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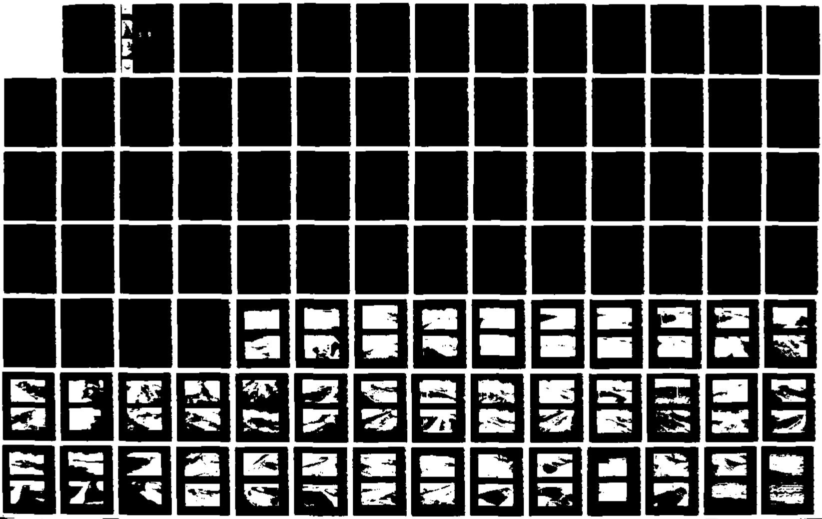

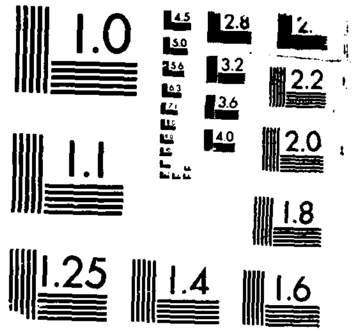

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US Army Corps of Engineers

TECHNICAL REPORT CERC-88-4

EFFECTIVENESS OF EXPEDIENT LEVEE-RAISING STRUCTURES

Experimental Model Investigation

by

Dennis G. Markle, Maury S. Taylor

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

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Levees	Sandbags	Flood control

19 ABSTRACT (Continue on reverse if necessary and identify by block number)

Experimental model tests were conducted at a scale of 1:1 (model to prototype) to define the static differential heads and dynamic (wave action) load limits beyond which selected existing Corps designs of expedient levee-raising structures will fail. Besides the existing designs, new concepts, submitted by various divisions and districts, were considered for testing as time and funding allowed.

Several designs, including 13 existing designs, 5 modified existing designs, and 4 new design concepts were tested and found to have varying degrees of success and failure under static head and wave action loadings.

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PREFACE

The study reported herein was authorized by the Office, Chief of Engineers (OCE), US Army Corps of Engineers, under the Improvement of Operation and Maintenance Techniques Civil Works Research Program Work Unit 31684. Funds for this work unit, titled "Effectiveness of Expedient Flood Fighting Structures," were provided through the Construction, Operations, and Maintenance Research Area under the field managership of the US Army Engineer Waterways Experiment Station's (CEWES's) Hydraulics Laboratory (HL) and Technical Monitors Messrs. J. L. Gottesman and C. W. Hummer, OCE.

The study was conducted at CEWES during the period April 1981 to June 1985 under the general direction of Mr. H. B. Simmons, former Chief, HL; Dr. R. W. Whalin, former Chief, Coastal Engineering Research Center (CERC); Mr. C. E. Chatham, Jr., Chief, Wave Dynamics Division, CERC; and Mr. D. D. Davidson, Chief, Wave Research Branch, CERC. The Wave Dynamics Division and its personnel were transferred from HL to CERC on 1 July 1983. Currently, Mr. F. A. Herrmann, Jr., is Chief, HL; Dr. J. R. Houston is Chief, CERC; and Mr. C. C. Calhoun, Jr., is Assistant Chief, CERC.

The study was conducted by Mr. M. S. Taylor, Engineering Technician, CERC, under the supervision of Mr. D. G. Markle, Research Hydraulic Engineer, CERC. This report was prepared by Messrs. Markle and Taylor and was edited by Ms. Shirley A. J. Hanshaw, Information Products Division, Information Technology Laboratory, CEWES.

Commander and Director of CEWES during the publication of this report was COL Dwayne G. Lee, CE. Technical Director was Dr. Whalin.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
inches	25.4	millimetres
miles (US statute)	1.609347	kilometres
pounds (force) per cubic foot	157.087476	newtons per cubic metre

EFFECTIVENESS OF EXPEDIENT LEVEE-RAISING STRUCTURES

Experimental Model Investigation

PART I: INTRODUCTION

The Problem

1. During high river stages, when the predicted maximum water levels encroach on or exceed the design freeboard of existing levees, it must be determined whether or not to construct a temporary levee-raising structure, and, if so, the type to be used. Although temporary structures, such as mud boxes, flashboards, sandbags, and potato ridges have been used as emergency measures along thousands of feet of levees in times of flooding, there are no documented load conditions for which the structures are assured to work. Many concepts are completely untried. During flooding, these expedient levee-raising structures have and will continue to cost vast amounts of public funds. They also will be expected to protect residential and commercial property and the individuals inhabiting these areas. Because of the possibility of catastrophic occurrences in the event of a structural failure, acquisition of data relative to expected performances during defined static (differential heads) and dynamic (wave attack) loadings is necessary.

Purpose of the Model Study

2. The purpose of the model study was to define the static and dynamic load limits beyond which selected existing US Army Corps of Engineers (Corps) designs of expedient levee-raising structures will fail. Besides the existing designs, new concepts submitted by various divisions and districts were considered for testing as time and funding allowed. Based on test results, recommendations for needed design improvements were made.

Approach

3. The tests were carried out in four stages as listed below:

a. Coordinating with Corps districts to obtain design and construction techniques used for existing expedient levee-raising structures and selecting concepts to be tested.

b. Testing of 2-ft-high structures.

c. Testing of 4-ft-high structures.

d. Testing of 6-ft-high structures.

Items a through d have been completed, and the test results are reported herein.

PART II: THE MODEL

Selection of Model Scale

4. In order to obtain an accurate evaluation of the various concepts being tested, it was necessary to reproduce strengths of various construction materials, strengths of the soils in the existing levee, proper flow nets induced by the differential heads, proper erosion rates, various wave characteristics, and other phenomena that occur because of the interaction of water with the various structures. State-of-the-art scale modeling does not lend itself to the accurate reproduction of all these phenomena at scales other than 1:1 (model to prototype). Therefore, all tests were conducted at a prototype scale using the same construction materials, construction techniques, and fluid media that exist in the prototype.

Test Facility and Equipment

5. Tests were conducted at a site adjacent to the Big Black River, approximately 10 miles* southeast of Vicksburg, Mississippi. Two test basins, one small and one large (Figure 1), were encircled by man-made levees representative of those found along the rivers and floodways of the Lower Mississippi Valley Division. Test sections were installed on the levee shared by the adjacent basins (Photos 1 and 2). Visual classifications of soil boring samples taken from this section of levee are presented in Table 1. A vertical displacement wave generator was installed in the small test basin (Photo 3) using an adjustable framework so the wave generator could be adjusted to the needed elevation for the various test water depths. Water was pumped from the Big Black River into the large test basin which was used as a water supply reservoir and as a means of capturing water released as a result of seepage through or failure of a test structure. The water level in the small basin was controlled by the use of a 2-cfs portable electric pump located adjacent to the test sections (Photo 1).

6. The existing levee was cut down to elevations 104, 102, and 100 ft

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

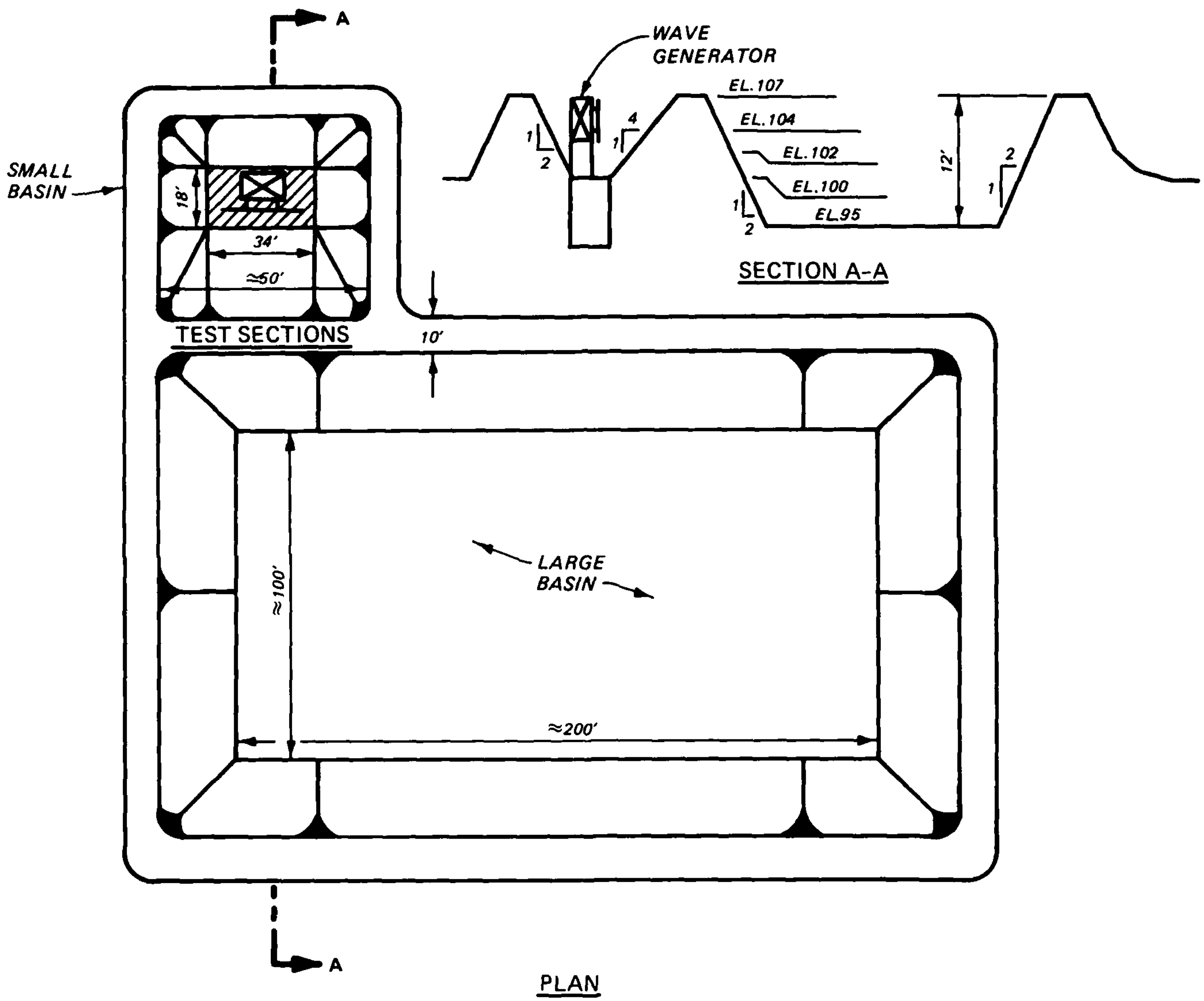

Figure 1. Test basin layout

for testing the 2-, 4-, and 6-ft-high structures, respectively (Figure 1). A fiber-reinforced rubber membrane was used to protect the cut area during periods when tests were temporarily stopped because of bad weather or flooding of areas surrounding the test site. During testing, the membrane was folded back (Photo 1) and served to protect the cut area and slopes of the existing levee from erosive effects of both seepage water and rapid release of water that occurred when a structure failed.

Construction of Test Structures

7. Because of the size of and restricted access to test structures, manual labor and hand tools were used to carry out tasks that would have been done ordinarily with heavy equipment during prototype construction. The test structures were constructed to reproduce, as close as possible, the quality of structure that would be constructed using conventional prototype equipment and techniques. Where wheeled or track-laying vehicles would be used to compact an earthen embankment in the prototype, a gas-operated vibrating compactor was used on the model test sections. To ensure that comparable compactions were achieved, measurements of in-place soil densities and moisture contents were taken on each test section (Photo 4), and these were compared to measurements taken on identical soil samples that were compacted by making passes with a conventional wheeled or track-laying vehicle. Additional construction details are provided in Part III.

Model Operation Procedures

8. The test section was installed in the cut area of the existing levee, and photographs were taken to document before-test conditions of the structure. Water was pumped from the large basin into the small basin until the desired water depth was reached in the small basin. The water level in the large basin was maintained around elevation 99 ft (Figure 1). The static water level was maintained in the small basin for a sufficient period of time to ensure that a flow net had adequate time to develop for the differential head being tested. If obvious seepage occurred through the structure, the differential head was maintained until such time that it was obvious seepage was not creating a stability problem or seepage resulted in failure of the

structure. If failure did not occur, the static water level was incrementally increased and maintained in the small basin until the structure failed or proved stable for the maximum static water level selected for testing of that particular structure. If the structure did not fail for the static differential head conditions, the water level was dropped to a predetermined elevation in the small basin; and the structure was exposed to cycles of wave attack. Afterward, the wave generator was stopped, and the structure was examined for any damage that had occurred.

9. Wave attack was continued until the structure failed or until it appeared no additional damage was occurring. If the structure did not fail at the initial water level, the water level was incrementally increased; and the structure was exposed to wave attack at each selected water level until the structure failed or proved to be stable for the maximum combined water level and wave attack. The condition of the structure was documented with photographs prior to changing from one test condition to another. At the end of the entire test series, the small basin was drained. The after-test condition of the structure was then documented with photographs presented herein. When the structure was being exposed to wave attack, waves were run in 15-min cycles during the 8-hr work day, and the test water level was maintained around the clock. During static differential head tests, the water level was maintained around the clock.

10. A summary of the tests performed on the various structures and the test results is provided in Table 2, while a detailed explanation is given in Part III.

PART III: TESTS AND RESULTS

Two-Foot-High Structures

Potato ridges

11. Flood-fighting manuals published by US Army Engineer Districts, Portland (CENPP) (1981), Omaha (CEMRO) (1978), Vicksburg (CELMK) (1975), and St. Paul (CENCS) (1973) recommend earth capping (potato ridges) existing levees as one option available to raise freeboard during times of predicted high water. Some districts state that the potato ridge should not be used where wave wash is possible, while others fail to state whether it should or should not be used in areas prone to wave wash. Although design and construction guidance is limited and varies from district to district, in general it is stated that a potato ridge's height is dependent upon the levee crest width and should not exceed 1.5 to 2.0 ft. The area of the levee crown on which the potato ridge is to be constructed should be scarified to provide for a good bond with the potato ridge material. The capping material should be dumped onto the levee crown by dump truck, spread in lifts (not to exceed 1.0 ft), and thoroughly compacted with a bulldozer by making passes with loaded trucks as the work proceeds.

12. Section 2-1. Section 2-1 (Plate 1 and Photos 5 and 6) consisted of a 2-ft-high potato ridge constructed of a locally available clayey silt. The riverside toe of the potato ridge was set back 1.5 ft from the riverside slope in the cut area of the existing levee. The area beneath the potato ridge (8 by 24 ft) was scarified using hand tools to reproduce the scarification that could be achieved with conventional construction equipment. The potato ridge was raised in 6- to 8-in. lifts and compacted with a gas-operated vibrating compactor (Photo 7). After the structure was between one-half and two-thirds completed and the last lift had been compacted, in-place moisture content (20.8 percent of dry weight, average) and soil density (average wet density = 110.1 pcf, average dry density = 91.2 pcf) measurements were made. Soil from the same borrow pit was placed on a scarified area adjacent to the test basins and was compacted by making five passes over the soil with a 2.5-yd rubber-tired bucket loader. In-place measurements of 18.4 percent dry weight, average moisture content, 117.4-pcf average wet density, and 98.4-pcf average dry density were made on the conventionally compacted soil. This

showed that the test section compaction was similar to, but did not exceed, the compaction that could be achieved using construction techniques outlined in the districts' flood-fighting manuals. Standard compaction and visual classification tests were conducted by the Geotechnical Laboratory (GL) at the US Army Engineer Waterways Experiment Station (CEWES) on a sample of the clayey silt taken from the same borrow pit (Plate 2).

13. Section 2-1 was exposed to 24 hr of 0.5-ft static differential head (Photos 8 and 9), 22 hr of 1.0-ft static differential head (Photos 10 and 11), and 6 hr of 1.5-ft static differential head (Photos 12 and 13). No noticeable seepage or stability problems were noted. Prior to starting and during the static head tests, the section was exposed to a cumulative rainfall of 4 to 5 in. which caused some noticeable surface erosion and promoted a good stand of nut grass on the structure. The nut grass began growing prior to starting the test and continued to grow above the waterline throughout the test. The grass roots probably prevented rain-induced surface erosion from being more severe.

14. Following static head tests, the water level was lowered in the small basin to the 1.0-ft static head water depth, and the structure was exposed to 19 hr of wave attack* over a period of 266 hr. The riverside slope of the structure eroded very rapidly during the first 8 hr of wave action. During this period, waves were breaking directly onto the structure's slope. This wave action produced rapid undermining which caused blocks of soil to fall from the upper slope (block failure). The erosion rate slowed considerably after the first 8 hr because of a berm of eroded material that formed causing waves to break prior to reaching the structure. Wave energy reaching the structure was reduced, and while the undermining and block failure rate slowed, they continued throughout the remainder of the test. Approximately 240 hr into the wave action test, seepage was noted on the landside of the structure; but by this time the crown of the section had completely disappeared, and the structure had eroded down the landside slope in some areas. Photos 14-20 show the condition of Section 2-1 at various points throughout

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* Because of the capabilities of the available wave generator, the wave height was limited to 0.4 ft during testing of all 2-ft-high structures. A larger wave generator was acquired for testing the 4- and 6-ft-high structures. It was capable of generating a maximum wave height of 0.75 ft. Both of these wave conditions are considered to be mild wave climates.

and at the end of wave action tests. Tests were stopped after 19 hr of wave attack. While the structure had not breeched, erosion was continuing at a very slow rate and would have eventually caused total failure if wave action had not been stopped. Photo 20 and two cross-sectional soundings (Plate 3) show the degree of erosion sustained by the compacted clayey-silt potato ridge.

15. Section 2-2. Section 2-2 (Plate 4 and Photos 21 and 22) consisted of a 2-ft-high potato ridge constructed of the same material and compacted (average moisture content = 17.5 percent dry weight; average wet density = 107.0 pcf; average dry density = 91.1 pcf) in the same manner as Section 2-1. The potato ridge was moved 1.5 ft riverward in the cut relative to the position previously occupied by Section 2-1, and the structure was covered with an opaque, 6-mil polyethylene (poly). The poly extended from 4 ft down the riverside slope of the existing levee to 1.5 ft beyond the landside toe of Section 2-2. Two sheets of poly were used along the length of the test section and were overlapped 3 ft at the center of the structure to simulate prototype conditions where two poly sheets were joined. Three methods of sandbag placement (as recommended in various district manuals) were used to hold the poly in place. These consisted of (a) laying the bags on the poly, (b) tying them to the poly, and (c) tying them at the opposite ends of a rope that extended over the potato ridge. Method (c) is recommended when placing the poly in standing water, while the other two methods, especially tying the bags to the poly, would have to be done before the water reached the area being protected.

16. The compacted clayey silt potato ridge had already proven to be stable for static differential heads up to 1.5 ft (Section 2-1), and the poly covering was added to see if it would provide adequate protection to prevent erosion resulting from wave wash. For this reason, Section 2-2 was not tested for static differential head conditions. The water depth in the small basin was raised to 1 ft relative to the toe of Section 2-2 and let stand for 24 hr to allow the clayey silt embankment to reach a static saturation level. The test section then was exposed to 34-1/2 hr of wave action over a period of 362 hr. The test section began to erode at the lap joint immediately after the wave action was started (Photo 23). The poly was more loose in this area, apparently allowing more wave energy to reach the embankment. Other portions of the riverside slope were eroding beneath the poly but to a much lesser

degree than in the lap joint area. After 2 hr of wave action, three sandbags were placed at the center of the riverside slope in an attempt to slow the erosion rate occurring at the lap joint (Photo 24). The sandbags helped for a short period, but these bags and all other sandbags exposed to wave action were emptied quite readily. It was obvious that the sand fill was too fine for the porous burlap bags when exposed to wave action. Unlike Section 2-1, whose crown and upper slopes remained firm and dry to the touch, Section 2-2 appeared to have a very spongy texture after the first few days of testing. The poly covering contained moisture, and large beads of condensed water were noticeable on the inside of the poly. Thus, even though the poly was reducing wave energy impinging directly on the structure, it was at the same time keeping the soil saturated (surface moisture could not evaporate as it would have on an unprotected structure, Section 2-1) which in turn made the soil more susceptible to erosion. Therefore, after 19 hr of wave action, it appeared that the erosion of Section 2-2 had progressed to approximately the same degree as had occurred on Section 2-1 after the same duration of wave attack. Photo 25 shows the test section after 10 hr of wave action and Photos 26 and 27 show the condition of the structure after 19 hr of wave action. The poly covering was removed (Photos 28 and 29) to take cross-sectional sounding (Plate 5) to compare to test results of Section 2-1. Comparison of Photos 20 and 28 and Plates 3 and 5 shows that both sections sustained comparable damage.

17. At the request of CELMK, the poly was placed back over the structure, and the test was continued to see if the poly would slow down the erosion rate once overtopping of the structure started. After a total of approximately 34 hr of wave action, a breech had formed in the test section; and water was overflowing the structure in this area and splashing over a large portion of the structure as each wave impinged on the test section. Test waves were continued for an additional 0.5 hr, at which point the test was stopped because the 1.0-ft water depth could not be maintained in the small basin. It was noted that once water started overtopping the structure, the poly reduced the erosion in the breech area to a much slower rate than would have occurred if water had been free flowing over the exposed soil. Between 34 and 34.5 hr of wave action, the poly separated at the lap joint in the breech, and the structure eroded at a very fast rate until the pump and wave generator were stopped and the water level in the small basin had dropped

below the breech elevation. Photos 30 and 31 show the condition of the test section after the small basin was drained. The structure was uncovered (Photo 32), and a cross-sectional sounding was made through the breech in the structure (Plate 6).

18. Section 2-3. Section 2-3 (Plate 7 and Photos 33 and 34) was a 2-ft-high potato ridge constructed of compacted clay gravel. The overall size, geometry, and construction of Section 2-3 were identical to those of Section 2-1, but the riverside toe of Section 2-3 was placed at the riverside edge of the levee cut as had been done with Section 2-2. Sections 2-2 and 2-3 were positioned as described at the request of CELMK. They stated that they did not recommend or use a setback (1.5 ft as used on Section 2-1) and that they were interested in seeing the tests conducted without the setback. In-place moisture content (8.7 percent of dry weight, average) and soil densities (average wet density = 121.0 pcf, average dry density = 111.3 pcf) were made on Section 2-3 when construction was one-half to two-thirds complete. Soil from the same borrow pit was placed on a scarified test area and compacted with a 2.5-yd rubber-tired bucket loader in the same manner as had been done with the clayey silt soil (described in paragraph 12). In-place measurements of 10.6 percent dry weight average moisture content, 124.5 pcf average wet density, and 112.5 pcf average dry density were made on the conventionally compacted clay gravel. These measurements again showed that the compaction achieved on the test section was similar to but did not exceed the level that can be achieved in the field using standard construction procedures. Standard compaction and visual classification tests (Plate 8) were conducted by the GL at CEWES on a sample of clay gravel taken from the same borrow pit. Visual classifications stated that the clay gravel was actually a gravelly, clayey sand, but it will be referred to as clay gravel for simplicity. The sample was scalped on a number 10 US standard sieve prior to running the laboratory compaction tests.

19. Section 2-3 was exposed to 24, 46, and 47 hr of 0.5-, 1.0-, and 1.5-ft static differential heads, respectively. Photos 35 and 36 show the condition of Section 2-3 at the end of the static head tests. The structure showed no seepage during the 0.5-ft static head test, but the landside toe did become spongy to the touch. Some very minor seepage occurred during the 1.0-ft static head test and increased for the 1.5-ft static head test. This seepage was an insignificant amount and did not cause obvious erosion or

stability problems. The structure was exposed to approximately 5 in. of rain during the static head tests which caused some minor surface erosion and accounts for a significant portion of the mud and water buildup on the landside of Section 2-3 (Photo 36).

20. Following the static head tests, the water level was lowered to 1.0 ft, and the structure was exposed to 28 hr of wave action over a period of 309.5 hr. Through a process of undermining and block failures, the first 5 to 6 hr of wave action produced rapid erosion of the clay gravel. During the erosion process, a graded beach was forming riverward of the section. Fine particles (clays, silts, and sands) were moved away by wave action, but the gravel was graded and deposited in front of the eroding embankment. The sloping gravel beach caused waves to break and dissipate a large portion of their energy; thus the erosion slowed considerably after 6 hr of wave action. As the gravel beach widened, erosive effects of the waves continued to diminish, and erosion was progressing at a very slow rate when the small basin was drained to document structure condition after 19 hr of wave action. Prior to draining the small basin, it was noted that the seepage rate had increased as the structure eroded away, but seepage had not yet caused any obvious stability problems. Photos 37-40 show the section condition after 2.5, 10.5, and 19 hr of wave action. The gravel beach is very noticeable in Photo 39.

21. The photos and cross-sectional soundings (Plate 9) show that Section 2-3 had accrued less damage than either Section 2-1 or Section 2-2 when exposed to the same duration of wave action. The small basin was refilled to the 1.0-ft water level. Wave action was continued to determine if the structure would fail from breeching of the crown, blow out because of seepage, or stabilize as a result of the gravel beach stopping the erosion process. After 28 hr of wave action, erosion had nearly stopped as a result of wave energy being dissapted on the gravel beach. The seepage rate had increased very slightly over the level noted after 19 hr of wave action but still was not causing any noticeable stability problems. (Quantitatively the amount of seepage was insignificant.) Photo 41 shows the condition of Section 2-3 after 28 hr of wave action at the 1.0-ft water level. The water level was raised to 1.3 ft, and wave action was continued to see if the structure would fail. Raising the water level so that waves could pass over the beach formed at the lower water level produced very rapid erosion. But after 12.5 hr of wave action run over a period of 142 hr at the 1.3-ft water level, the beach had

adjusted to the higher water level and once again had stopped erosion. The small basin was drained, and the structural condition was documented with Photo 42 and cross-sectional soundings (Plate 10). Photo 43 shows a close-up of the gravel and sand beach that formed on the riverside of the eroded section.

22. The small basin was refilled to a water level of 1.4 ft and wave action was continued. After 15 min of wave action at the 1.4-ft water level, a small breech was produced in the crown of the section. After an additional 22 min of wave action (37 min total) the breech had opened to such a degree that the 2 cfs pump could not maintain the water level in the small basin. At that point, the wave generator and pump were shut off, and water continued to flow through the breech and erode the structure until the water level in the small basin equalized with the cut elevation through the structure. The remaining water then was pumped from the small basin. Photographs 44-46 and a cross-sectional sounding (Plate 11) through the structure's breech show the condition of Section 2-3 at the end of the test.

23. Section 2-4. Section 2-4 (Plate 12 and Photos 47 and 48) was a new design submitted by and tested at the request of CELMK. The structure consisted of a 2-ft-high compacted clayey silt potato ridge encased in 6-mil opaque polyethylene. In lieu of scarifying the surface of the existing levee, a seepage key was incorporated into the design to deter piping of water underneath the structure. Unlike Section 2-2, only one sheet of poly was used; therefore, the lap joint at adjoining sections of poly was not reproduced for this test. It should be noted that a poor lap joint construction could result in instability that was not observed during this test. In-place moisture content (17.2 percent dry weight, average) and soil densities (average wet density = 100.5 pcf; average dry density = 85.7 pcf) measurements were taken when the structure was approximately two-thirds complete. At the request of CELMK, this clayey silt structure was not as thoroughly compacted as previously tested clayey silt structures.

24. Section 2-4 was exposed to 24, 95, and 24.5 hr of 0.5-, 1.0-, and 1.5-ft static differential heads, respectively, and no significant seepage or stability problems were observed (Photos 49 and 50). By the end of the static head tests, the test section was spongy. As in Section 2-2, the poly trapped the moisture that would have otherwise dissipated through evaporation. Without evaporation, the structure became saturated with water that entered

the test section around the outer edges of the poly. Water condensed inside the poly and dripped back onto the structure. This condensation kept the upper portions of the structure saturated; whereas without the poly covering the surface areas would have remained comparatively dry. The poly cover did prevent surface erosion that would have occurred after 2 to 3 in. of rain that fell during the static head tests.

25. Following the static head tests, the water level in the small basin was lowered to 1.0 ft, and the structure was exposed to 19 hr of wave action over a period of 266.5 hr. Erosion of the soil in the wave action zone beneath the poly started immediately. The erosion rate slowed as the test progressed and had all but stopped by the 19th hour of wave action. The small basin was drained to document the condition of the test section (Plate 13 and Photos 51 and 52). The eroded material was contained by the poly but had produced a large bulge on the riverside toe and had stretched the poly very tight. Tension cracks were appearing along the crown of the structure, and the riverside of the crown was being vertically compressed due to forces being applied by the stretched poly covering. The poly had not torn loose on the landside toe, but it was showing signs of stretching and tearing at the points where the anchor pins projected through the poly. It was noted throughout wave action at the 1.0-ft water level that minor amounts of material were being pumped out of the structure through 2 or 3 small holes (diameters equal to approximately 1/4 in.) in the poly on the lower riverside slope. This material loss was insignificant, but it did show that material could be pumped through the holes by wave action; and if the holes were large enough, the material loss could have been significant. It is surmised that large losses of material would have eliminated the bulge that occurred on the riverside toe and would have kept the poly from stretching tight. Thus the protection provided by the bulge and tight poly covering would have been reduced, and the erosion action would have been more rapid and extensive resulting from large amounts of wave energy reaching the uneroded portions of the structure.

26. After 19 hr of wave action at the 1.0-ft water level, no significant seepage problems were evident, but the structure continued to be saturated and spongy. The water level in the small basin was raised to the 1.3-ft level, and the structure was exposed to 25.5 hr of wave action over a period of 288 hr. During the first 3 hr of wave action, a small sump pump had to be run to replace the water being lost due to overtopping which stopped when the

eroded material increased the height of the bulge in the poly toe. This increased height tripped the wave farther riverward and thus dissipated the wave energy that had previously produced the wave overtopping. After 14.5 hr of wave action, the poly had pulled loose halfway across the landside toe (Photo 53). The poly continued to be torn loose and move riverward as the material in the bulge on the riverside toe migrated down the slope of the existing levee. After 18 hr of wave action, the poly had been pulled over the crown on a portion of the structure (Photo 54). From this point on, the erosion rate became more rapid. After 25.5 hr of wave action, the crown was gone on the center one-third of the structure (Photo 55), and water was overtopping the structure during wave action. The structure was left for the weekend with a 1.3-ft water level in the small basin, and at some time during the weekend the structure breeched. The remaining water was pumped from the small basin, and the condition of the structure was documented (Plate 14 and Photos 56 and 57).

Earth-filled sacks

27. The use of earth-filled sacks (sandbags) is very common in flood-fighting construction. Their use is recommended for consideration in all of the flood-fighting manuals reviewed. In general, structures properly constructed with sandbags are easy to construct and perform quite well; but they require more manpower and time to construct than many other levee-raising structures.

28. Section 2-5 (Plate 15 and Photos 58 and 59) was a 2-ft-high structure constructed of earth-filled sacks. The structure was constructed using the design and construction techniques outlined in the CELMK "Flood Emergency Handbook" (1975). The structure was 6 ft wide at the base, 2.5 ft wide at the crown, and set back 1.0 ft from the riverside edge of the existing levee. Prototype construction guidance recommends removal of sod from the area where the sandbag structure will rest. No sod existed on the test levee, but the area was cut down to a depth of approximately 0.2 ft to simulate removal of sod, and the bottom layer of sandbags was placed in this cut. This cut aided in bonding the sandbags to the existing levee and helped prevent piping of water underneath the structure. Seams of adjacent rows and layers were offset to break seepage lines. The sacks were one-half to two-thirds full and were not tied shut. The untied end was folded back, and the closed end of the next sack was placed over the fold. The sacks were tamped into place by walking

over the sacks to increase the density of the structure. It took seven layers of sandbags to reach a height of 2.0 ft, and the layers were stair-stepped as shown on Plate 15. The stair stepping adds stability and aids in breaking the joints between successive layers. Depending on the size of sacks used, the number of layers needed and the setback used for successive layers will vary. Loosely woven burlap sacks, 15 by 30 in. (provided by CELMK), were used to construct Section 2-5. The two soil types (clayey silt and clay gravel) previously used to construct Sections 2-1 through 2-4 were selected to fill the sacks. Facing the riverside of the test section, the left half of the structure consisted of clay gravel-filled bags; and the right half of the structure consisted of clayey silt-filled bags. The test results could thus provide a comparison of how prone the two materials are to leaching through the openings of a loosely woven sack.

29. Section 2-5 was exposed to 240, 24, and 24 hr of 0.5-, 1.0-, and 1.5-ft static differential heads, respectively. Two to three inches of rain fell during the static head tests. The clayey silt-filled bags felt spongy, whereas the clay gravel-filled bags felt firm. This was the only noticeable difference between the two structure halves. The rate of seepage increased with increasing differential head, but the amount of seepage was insignificant and did not cause any leaching of material out of the landside bags.

30. Following the static head tests, the structure was exposed to 19 hr of wave action over a period of 191 hr at the 1.0-ft water level (Photo 60). Emptying of the clayey silt-filled bags in the wave action zone started immediately and progressed at a rather rapid rate for the first 3 to 4 hr of wave action. The clay gravel-filled sacks showed some emptying, but it was at a slower rate and to a lesser degree. During the 19th hour of wave action, all noticeable leaching of fill material had stopped, and the small basin was drained. Photos 61 and 62 and cross-sectional soundings (Plate 16) show the condition of Section 2-5 after 19 hr of wave attack. The clayey silt-filled sacks had eroded back and formed a vertical face at the riverside of the structure crown. The clay gravel-filled sacks showed some emptying, but it was to a much smaller degree than that of the clayey silt-filled bags.

31. The small basin was flooded to the 1.3-ft water level, and the structure was exposed to 19 hr of wave action over a period of 384 hr. The structure was exposed to 4 to 5 in. of snow and 0.5 in. of ice during this time period, and the wave action was halted for a week because ice had formed

in the small basin. The clayey silt-filled sacks showed rapid erosion during the first 6 hr of wave action, but then the erosion rate slowed and progressed at a slower rate throughout the remainder of the test. The clay gravel-filled sacks showed some additional damage during the first few hours of wave action at the 1.3-ft water level, but no noticeable erosion occurred during the final 3 to 4 hr of wave action. It is felt that once the fines were leached out of the riverside of the clay gravel-filled bags, the coarser fill material acted like a wave absorber and filter which prevented additional leaching of the fines. Although the seepage rate through the structure increased with increasing damage to the structure, the amount neither became significant nor appeared to cause any stability problems. The seepage rates appeared to be equal on both halves of the test section. The test was stopped after 19 hr of wave action at the 1.3-ft water level. Photos 63 and 64 and cross-sectional soundings (Plate 16) show the condition of Section 2-5 at the end of the test.

Four-Foot-High Structures

Earth-filled sacks

32. Section 4-1 (Plate 17 and Photos 65 and 66) was a 4-ft-high structure constructed of earth-filled sacks using design and construction techniques outlined in the CELMK "Flood Emergency Handbook" (1974). The test section was 12 ft wide at the base, 2.5 ft wide at the crown, and set back 1.0 ft from the riverside edge of the existing levee. The levee was prepared for placement of the earth-filled bags by cutting a trench 0.2 ft deep and 12 ft wide along the approximate 24-ft length where the sacks were placed. As discussed in paragraph 28, this provided for a bond between the existing levee and earth-filled sacks. Section 4-1 was constructed in the same manner as Section 2-5 (paragraph 28). Facing the riverside of the test section, the left side was constructed with clay gravel-filled, 15- by 30-in. loosely woven burlap sacks (provided by CELMK). The center and right-hand sides were constructed using clayey silt-filled, 19- by 35-in. spun woven polypropylene and 21- by 36-in. woven polypropylene sacks, respectively (provided by US Army Engineer District, New Orleans, (CELMN)). The polypropylene sacks had a much tighter weave than the burlap sacks. For this reason, a clayey silt fill was used in the polypropylene sacks. If this material did not leach out of the sacks, coarser fill material would be even less susceptible to leaching. The

test results also provided a comparison between the clayey silt-filled burlap sacks on Section 2-5 and the clayey silt-filled, spun woven, and woven polypropylene sacks on this section.

33. Following completion of the structure, tests were shut down for approximately 3 months because roads leading to the test site were flooded. During this time the structure was exposed to approximately 20 in. of rain. The burlap sacks began to show signs of deterioration which probably resulted from the combined effect of rains and ultraviolet radiation. The woven and spun woven polypropylene sacks showed no evidence of deterioration.

34. Section 4-1 was exposed to 750 hr of between 1.1- and 2.0-ft static differential head. This long duration was the result of being flooded out of the test site for approximately 1 month after the start of the 2.0-ft static head test. Except for the continued deterioration of the burlap sacks, the structure looked very good at the end of this portion of the test (Photos 67 and 68). There was no evidence of leaching of sack-fill material and, even though seepage through the structure was more pronounced on the polypropylene sacks than on the burlap sacks, the amount of seepage was very minor. The static differential head was raised and maintained between 3.0 and 3.35 ft for 48 hr. Seepage through the section increased slightly, but no material leaching was evident. Photos 69 and 70 show the saturation level and seepage through the structure at the end of the static head tests. In summary, Section 4-1 was in good condition at the end of the static head tests (Photos 71 and 72) and was providing its intended protective function. The burlap sacks showed some moderate deterioration, and the spun woven polypropylene showed some minor weathering. The test section appeared to have settled approximately 0.1 to 0.2 ft in the area where the burlap and spun woven polypropylene sacks interfaced, but this had not affected the function of the test section.

35. The test basin was flooded to a 1.0-ft depth, and Section 4-1 was exposed to 19 hr of wave action* over a period of 264 hr. The burlap sacks sustained the most damage with broken sacks below and just above the still-water level (swl). The woven polypropylene sacks showed some damage but to a lesser degree than the burlap sacks and the spun woven polypropylene sacks which had sustained only very minor damage (Plate 18 and Photos 73-77). The

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* Because of the capabilities of the available wave generator, wave height was limited to ~0.75 ft during testing of all 4-ft- and 6-ft-high structures.

clayey silt fill appeared to be leaching out of the folded ends of the woven sacks. The structure was still in good condition, but the damage had not subsided when the wave action was stopped at the 1.0-ft depth. The test basin was flooded to the 2.0-ft depth, and Section 4-1 was exposed to 19 hr of wave attack (Photo 78) over a period of 670 hr. Considerable amounts of rain fell during this time, and, once again, the test site was flooded. At the end of wave attack at this water depth, the damage rate had become very slow. The structure had sustained considerable damage, but it still was providing good protection to the landside of the structure (Plate 18, Photos 79 and 80). The woven polypropylene and burlap sacks had continued to empty in the same manner as had been observed during wave attack at the 1.0-ft depth. The spun woven polypropylene sacks showed the highest degree of damage at this point in the test. The thread used by the manufacturer to sew the sacks together had deteriorated and broken, and this resulted in rapid emptying of several sacks (Photo 81). The structure was then exposed to 8 hr of wave action over a period of 116 hr at a 3.0-ft water depth (Photo 82). The structure sustained additional damage at this test condition but was still in relatively good condition at the end of the test (Plate 18 and Photo 83), and it could have withstood several more hours of this test condition before its stability would have become questionable. It should be noted that a large portion of the material on the lower half of the burlap sack section in Photo 83 was eroded off the existing levee and deposited in this area. Minor to moderate amounts of wave overtopping occurred at the interface of the burlap and spun woven sacks (Photo 84), but this did not cause any measurable amount of damage on the landside of the test section.

Plastic grid with sand fill

36. Section 4-2 (Plate 19 and Photos 85-87) was a 4-ft-high structure constructed of six lifts of plastic grids filled with masonry sand. This was a new concept inspired by the expedient road construction techniques research being carried out by GL at CEWES. For road construction, the expandable plastic grid is placed in 8- by 20-ft sections, filled with sand, and overlayered with asphalt. Considerable time and effort have been expended by GL in optimizing the geometry, size, and construction materials for the plastic grid in order to provide maximum strength, ease of shipping and handling, and minimum cost. During testing, GL found that the light-colored plastics are highly susceptible to deterioration when exposed to ultraviolet light for extended

periods of time and that the use of dark-colored plastics minimizes this problem. Presently, there is only one manufacturer set up to produce the plastic grids, and 6 weeks delivery time is the minimum for any order.

37. In order to expedite the Coastal Engineering Research Center's (CERC's) testing program, GL supplied CERC with some surplus white medium-strength plastic grids which were cut down to the sizes needed to construct Section 4-2, (Photo 88). A trench 0.2 by 3.2 by 24.0 ft was cut in the existing levee 1 ft landward of the riverside edge. The structure was constructed in six 0.67-ft-high lifts with offset joints between successive lifts. Each section of grid was expanded to its predetermined length and width and staked in place (Photo 89). This length and width maximized the volume of each cell, thus requiring a minimum number of cells to construct the test section. The sand fill was dumped on each lift (Photo 90), and the excess was screeded off (Photo 91). No effort was made to compact the fill material. Facing riverside of the test section, the left side had spun-woven filter fabric between lifts (Photo 92), the center had no material between lifts (Photo 93), and the right side had burlap sacks between lifts (Photo 94). The burlap and filter fabric extended approximately 1.2 ft (one and one-half cell depths) back into the section (Photo 95). This material was used to see if it would enhance stability of the test section. (Filter fabric had been used between successive lifts when limited tests were done at CEWES to determine the feasibility of using sand-filled plastic grids to construct military bunkers).

38. Several difficulties were encountered during construction which slowed assembly speed for this type of test section. The major problems encountered were as follows:

- a. Cold weather caused the grid to be stiff, making it difficult to expand and maintain uniform cell sizes.
- b. Even after warming the grids, it was difficult to maintain uniform cell size.
- c. It was difficult to get outside cells on the riverside and landside to line up exactly in order to keep the sand fill from leaching out (some small pieces of plastic were used in these areas where filter fabric or burlap was not used to aid in keeping the sand in the cells).
- d. Some cell breakage occurred during expanding and filling of the grids.

Cell breakage, which occurred on the outside cells adjacent to the welds, could have resulted from an inherent weakness in the white plastic. It was

not known how much ultraviolet light exposure had occurred on the plastic grids prior to CERC's receipt from GL. The plastic grid weakness became quite apparent later in the tests.

39. Section 4-2 was exposed to 24, 96, and 24 hr of 1.0-, 2.0-, and 3.0-ft static differential heads, respectively. The structure performed quite well for all conditions, allowing only minor amounts of seepage to reach the landside toe. The test section did sustain increasing degrees of damage corresponding to the increases in static differential head. Immediately after flooding to the 1.0-ft depth, the sand fill subsided from 1/2 to 2-1/2 in. in the outside top cells in the middle of the riverside of the test section and 1/4 to 2 in. along the entire outside top row on the landside of the test section. The subsidence was a combined effect of consolidation of the sand in the cell columns due to saturation in the lower lifts and loss of sand in areas where the cells in successive lifts did not line up exactly. The cells with either burlap or filter fabric between lifts showed no subsidence of the sand fill. All damage had subsided prior to raising the static differential head to the 2.0-ft level. At the end of the 2.0-ft static head the seepage had increased slightly in the middle of the test section, but all damage had ceased (Photos 96-100).

40. The middle of the section on the riverside had accrued moderate damage. Several of the outside rows of cells were emptied down to the fourth lift from the bottom. The sand fill in the outer row on the middle of the landside of the test section had subsided a maximum of halfway down the top lift. The sand fill in various cells on the remainder of the test section showed some additional subsidence during the 2.0-ft differential head, but it was very minor in comparison to the areas of the test section that had no filter fabric or burlap between lifts. During the 3.0-ft static head test, the sand fill in several cells on the riverside in the middle section, one in the middle, and several in the first row on the riverside of the burlap section showed additional lowering. All damage had stopped by the end of the 3.0-ft static head, and the seepage had only increased slightly on the middle section.

41. Heavy rains occurred during the 2.0- and 3.0-ft static head tests. By the end of the static head tests we were forced out of the test site for one month by flooding. Prior to leaving the test site, the test section was covered with black 6 mil polyethylene in an effort to protect the plastic grid

from ultraviolet radiation. Upon returning to the test site, we found that high winds had blown a large portion of the black polyethylene off the test section. After uncovering the remainder of the section, we found that approximately one-third of the outside row of cells in all six lifts on the landside and approximately ten cells on the riverside of the test section had broken adjacent to the welds which held the cells together. The riverside cell breakage was mainly concentrated in the middle of the test section, and it caused significant damage on the riverside of the test section.

42. Rainfall had filled the small basin to a depth of approximately 0.5 ft. The depth was increased to 1.0 ft, the section was exposed to 0.5 hr of wave action (Photo 101). The condition of the test section was then documented (Photos 102-106). After 19 hr of wave action at the 1.0-ft level over a period of 168 hr, the middle section had sustained significant damage on both the riverside and landside. The burlap and filter fabric sections still were in good condition on the riverside, but cell breakage had continued to occur on the landside throughout this portion of the test. Seepage through the structure had increased slightly, but it was still an insignificant amount. All damage resulting from wave action appeared to have stopped, and Photos 107-111 show Section 4-2 at the end of wave action at the 1.0-ft depth.

43. The basin was flooded to the 2.0-ft depth. At the start of wave action (Photo 112), a moderate amount of seepage was occurring at the interface of the filter fabric and unfiltered section (Photo 113). The seepage would decrease when sand would fall from the upper cells down to the riverside toe and would slowly increase as the sand was eroded away by wave action. When wave action was stopped to document conditions of the test section, seepage would decrease, indicating that wave action was causing increased seepage. After 2-1/2 hr of wave action, seepage had increased to the point that, even with a small sump pump running, the small basin would drop approximately 0.2 ft every 0.5 hr (Photo 114). The small basin was drained, and the condition of the test section was documented (Photos 115-117). The middle section showed significant damage which had not subsided, but there was a continuous row of cells across the middle section that was full of sand up through the fifth lift. For this reason, it was decided to flood the basin to a 3.0-ft depth and see how long it would take to fail the section with wave action at that level. The middle of the section failed at the interface of the filter fabric and unfiltered section when the water level reached 2.4 ft (Photos 118

and 119). The middle section failed completely. The remainder of the test section (burlap and filter fabric sections) was in relatively good repair, except for the landside cell breakage, and was still providing good protection. It could most likely have withstood quite a bit more wave action at either the 2.0- or 3.0-ft water depth. Photographs 120-122 show the condition of Section 4-2 after failure.

Plywood flashboard

44. Several of the flood-fighting manuals published by Corps districts contain designs for plank and/or plywood flashboards. These structures are recommended as one alternative for providing additional freeboard and protection against static heads when a levee needs to be expeditiously raised more than 2.0 ft above its existing elevation. No mention is made of their use in wave action zones, but it seems to be implied. CELMK (1975) has the only recommended design for a flashboard that has no earth or sack backing.

45. Section 4-3 (Plate 20 and Photos 123 and 124) was constructed of a plywood flashboard whose design was extracted from the LMK Flood Emergency Handbook (1975). The section was constructed using the following procedures:

a. A trench, 1.0 ft deep, 0.5 ft wide, and 24 ft long, which was parallel to and set 1.0 ft back from the riverside edge of the crown, was cut in the existing levee (Photo 125).

b. Holes were dug; and 6.5-ft-long, 4- by 4-in. wide, wooden posts were set on 3-ft 9-in. centers against the landside of the trench and extended 3 ft below the surface of the existing levee (Photo 126). (In the prototype, the post would most likely be driven into the levee. Access to the test site did not allow the use of large equipment to drive the posts. For this reason a post hole digger was used, and the earth was tamped back into the hole around the posts. If anything, this provided for a slightly weaker structure than would have been attained by driving the post. Thus our test structure was on the conservative side.)

c. Sheets of exterior plywood, 4- by 8-ft by 3/4 in., were nailed to the posts, and 6-in. glued lap joints were used at every other post.

d. The existing levee soil was tamped into the trench.

e. Stakes (2 by 4 in. boards), 2-1/2 ft long, were driven 2.0 ft into the existing levee a distance 6 ft behind the posts, and 8-ft-long, 2- by 4-in. boards were nailed between the posts and the stakes.

46. When the water depth had reached 0.95 ft during flooding to the 1.0-ft static differential head level, seepage boils began around the posts of

Section 4-3 (Photo 127). The 1.0-ft static differential head was left on the structure overnight. At some time during the night, the structure failed. The seepage flow had removed the trench fill material around one post and along a length of the structure that encompassed three consecutive posts. The small basin was flooded back to the 1.0-ft depth, and the filling pump was left running in order to document the flow under the structure at this 1.0-ft static differential head (Photo 128). Photographs 129-131 show the condition of Section 4-3 after its failure at the 1.0-ft static differential head.

Plywood flashboard with earth backing

47. The CELMK Flood Emergency Handbook recommends for high or extended periods of static differential that either earth or earth-filled sack backing be placed on the landside of the plywood flashboard (Section 4-3). No recommendation is given concerning the amount or geometry of earth or sack backing. It was noted at the end of testing of Section 4-3 that the wood structure was still in very good condition. Therefore, it was decided to repair the trench and place a tamped earth backing on the landside of the test section to see if this would improve its performance when exposed to static differential heads.

48. Section 4-3-A. The 2.0- by 4.0-in. by 8.0-ft braces were removed from Section 4-3, and an area 4.0 ft wide on the landside of the plywood and extending the full length of the structure was plowed (scarified) with hand tools to provide a bond between the existing levee and the earth backing. Compacted clayey silt fill was placed behind the section, and the braces were nailed back in place. This section was referred to as Section 4-3-A (Plate 21 and Photo 132). The method of constructing the clayey silt fill was identical to that used to construct Section 2-1 (described in paragraph 12). The clayey silt fill was 2.0 ft high with a crown width of 1.0 ft and a landside slope of 1V on 1.5H. In-place soil density and moisture content were not measured, but they should have been very similar to values measured on Section 2-1.

49. Section 4-3-A was exposed to 24 hr each of 1.0-, 2.0-, and 2.9-ft static differential head. The structure performed very well for all these test conditions. The only seepage occurred where the ends of the section tied into the existing levee. This area was not considered in the test, but the seepage was minimized by placing sandbags in the areas experiencing seepage. This prevented failure of the structure due to end effects that were not being considered in the test. Photographs 133 and 134 show the condition of Section 4-3-A at the end of the 2.9-ft static differential head. The water level

was lowered to a 1.0-ft-depth, and the structure was exposed to 7.25 hr of wave action over a period of 48 hr (Photo 135). The structure showed no signs of damage, and no water overtopped the plywood during wave action at this water depth. The water depth was increased to 2.0 ft, and wave action was started (Photo 136). Wave action at this water level produced occasional overtopping (Photo 137). This overtopping water was causing minor amounts of surface erosion on the clayey silt fill when the tests had to be shut down due to lack of funds for the remainder of fiscal year 1983 (FY 83). The basin was drained before leaving the test site.

50. During the 2-month delay in testing, a dense growth of nut grass became established on the earth backing of Section 4-3-A (Photo 138). The small basin was filled to a depth of 2.0 ft, and wave action was restarted. After 5.0 hr of wave attack over a period of 72.0 hr, the earth backing showed very minor surface erosion caused by occasional overtopping water. It was noted that the nut grass growth served to slow down the erosion rate, and it was felt that on a typical flood-fighting structure sufficient time would not exist for a vegetative covering to develop. Therefore, prior to raising the water level to 2.5 ft and continuing wave action, the grass was removed from a portion of the earth backing (Photo 139). Wave action at the 2.5-ft water level produced considerable, but not continuous, wave overtopping (Photo 140). Sixteen hours of wave attack over a period of 312 hr produced moderate erosion of the earth backing where the nut grass had been removed. The overtopping waves lowered the crown a maximum of 0.5 ft in some areas, and surface erosion steepened the landside slope (Photos 141 and 142). The eroded portion of the earth backing exhibited a slight increase in seepage rate as the erosion progressed, but the amount of seepage was not significant and did not appear to be contributing to the erosion of the earth backing. Most of the seepage noted was occurring around the 4- by 4-in. support posts. The remainder of the structure showed little or no erosion. The overtopping water would have eventually caused the section to fail, but it was felt that with minimal maintenance the section could have withstood the test conditions for an indefinite period of time.

51. <u>Section 4-3-B.</u> The compacted clayey-silt earth backing on Section 4-3-A was cut down to an elevation of 1.0 ft to determine if a smaller amount of compacted earth backing would provide the same protection as the large amount of backing used on Section 4-3-A. This structure was referred to

as Section 4-3-B (Plate 22 and Photo 143). The test section was exposed to 96 hr of 1.0-ft static differential head, and no seepage was observed. After increasing the differential head to 1.5 ft, small amounts of clear seepage started around two support posts. After 24 hr, seepage was occurring at all of the support posts (Photo 144). The seepage was clear as it exited around the posts, and the seepage rate did not appear to be increasing with time. The static head was increased to 2.0 ft which resulted in increased seepage around the support posts and some surface erosion on the crown and landside slope of the earth backing. After 24 hr of 2.0-ft static head (Photo 145) the seepage rate had not increased; and, except for some superficial surface erosion, the section was in good condition. The static head was increased to 2.5 ft which resulted in seepage around one of the center posts developing into a small water spout that appeared to be transporting some material. After 24 hr the seepage rate had not increased, and the water spout appeared to have stopped transporting material. The earth backing had become very saturated, and seepage was continuing to cause surface erosion (Photos 146 and 147). Increasing the static head to 2.9 ft caused the water spout to increase and once again start to move material. At some point during the night, the earth backing failed around the support post that had exhibited the worst seepage and at the toe adjacent to this same support post (Photos 148-150).

52. Section 4-3-C. The remainder of the compacted earth backing was removed from Section 4-3-B. The damaged area of trench was cut back, refilled, and compacted, and a loose earth backing was shoveled onto the landside of the plywood flashboard to an average elevation of 2.0 ft. In-place wet density, dry density, and percent moisture content of the earth backing averaged 85.0 pcf, 74.8 pcf, and 15.5 percent, respectively. This structure was referred to as Section 4-3-C (Plate 23 and Photo 151). The section was tested to compare the protective capabilities of the loose earth backing on the structure to those of the compacted earth backings used on Sections 4-3-A and 4-3-B. After 24 hr of 1.0-ft static head there was no measurable seepage, but there were very obvious wet areas on the landside slope (Photo 152). The static head was raised to 1.5 ft, and after 24 hr the majority of the landside slope and crown were wet. Some very minor seepage was noted along the toe of the earth backing, but no movement of earth was obvious (Photo 153). The 1.5-ft static head was maintained for an additional 24 hr to see if seepage rate would increase and possibly cause stability problems. After 48 hr of

1.5-ft static head, except for one small area, the entire earth backing surface had become damp. The only area saturated was along the landside toe where the seepage line exited the earth backing.

53. The seepage rate had not increased, and there were no obvious stability problems (Photo 154); therefore the static head was increased to 2.0 ft. Heavy rains occurred during the 72 hr that the 2.0-ft static head was maintained. The rain caused some surface erosion and consolidation of the earth backing. Areas of the crown were lowered up to 0.5 ft. There was obvious seepage along the saturated landside toe, but the amount was insignificant and was not moving material (Photos 155 and 156). The static head was increased to 2.5 ft, which resulted in seepage where the center support posts exited the crown of the earth backing and a noticeable increase in seepage along the landside toe (Photo 157). After 24 hr, seepage continued around the center support posts and landside toe; but the seepage water was clear, indicating that the seepage was not producing erosive action (Photo 158). Seepage around the center support post was flowing down the landside slope; but because of the small volume of water, surface erosion was very minor (Photo 159). While static head level was being raised to 2.9 ft, leakage around one support post became quite severe. This flow eroded the earth backing and led to failure of the structure (Photo 160). Photos 161-164 show the condition of the test section after draining the test basin. Although the earth backing had failed, the wood structure had sustained no structural damage.

54. Section 4-3-D. The remainder of the earth backing was removed from Section 4-3-C; and the damaged area of the trench was cut back, refilled, and compacted. In an effort to see if a sealer could be used to replace the earth backing, the riverside and landside of the structure were sprayed with a thick coating of slow setting emulsified asphalt (Plate 24 and Photo 165) and was left to cure overnight. This was referred to as Section 4-3-D. During this period, heavy rains occurred which washed away the majority of the asphalt (Photos 166 and 167). The structure was resprayed with rapid setting emulsified asphalt and allowed to cure for 3 days (Photos 168 and 169). During this period light rains occurred, but they did not appear to affect the asphalt coating. A static differential head of 1.0 ft was placed on the structure, and after 72 hr there were no obvious seepage problems. The static head had lifted the landside asphalt coating in some areas, and small amounts of

seepage could be seen exiting from beneath the asphalt; but the water was clear (Photo 170). The static head was increased to 1.5 ft and maintained for 24 hr. During this period heavy rains occurred. The seepage increased slightly, but the structure was still functioning very well (Photo 171). The static head was increased to 2.0 ft, and a slight increase in the seepage rate was noted; but the quantity of seepage was still minor. After 24 hr of 2.0-ft static head, the structure was still in good condition. However, it had become obvious that the majority of seepage was coming from around a support post. This became apparent when a prominent bulge appeared in the asphalt coating around the post (Photo 172). The seepage around this post became significant and began moving material when the static head was increased to 2.5 ft. After 2.0 hr the water broke through the asphalt and began free flowing under the plywood flashboard (Photo 173). Photo 174 shows where the asphalt coating and compacted earth-filled trench failed around one of the center support posts.

Planking flashboard with earth backing

55. CELMN (1983), CELMK (1975), CEMRO (1978), and CENPP (1981) recommend a planking flashboard with earth backing when a required levee capping height exceeds 1.5 ft but is less than 3 ft or if the capping is likely to be exposed to wave action. Recommended construction techniques vary slightly from district to district, but in general end products are the same. Generally, construction guidelines call for a furrow to be plowed approximately 1.5 ft from the riverside of the crown in a direction that deposits soil toward the landside. The furrow should be as straight as practical and at least 2 in. deep. Some districts state that you can place the horizontal planking on either side of the driven 2- by 4-in. support posts. When the planking is placed on the riverside, boards should be placed along the plowed furrow and then the 2- by 4-in. support posts driven in such a manner that they jam the planking against the vertical face of the plowed furrow. This method has the advantage of the jam fit on the bottom board. The advantage of placing planking on the landside of the support posts is that pressure created by the earth backing is transferred directly to the support posts. Some districts state that all vertical joints in the planking should fall at the support posts, while others say that either scab or butt joints can be used at any point along the structure.

56. <u>Section 4-4.</u> Section 4-4 (Plates 25 and 26 and Photos 175 and 176) was a 3-ft-high planking flashboard with earth backing. A furrow was plowed approximately 1.5 ft from the riverside of the levee crown, and 5-ft-long, 2- by 4-in. posts were driven to a depth of 2 ft spaced on 4-ft centers along the riverside edge of the plowed furrow (Photo 177). Then 1- by 12-in. planking was nailed three high to the landside of the support posts using 20d common nails that were clinched on the riverside. Both scab and butt joints were used at random along the 24-ft-long structure (Photo 178). A single thickness of burlap sacking material was stapled to half of the flashboard on the landside, while the other half was covered with black, 6 mil polypropylene. The existing levee surface was scarified prior to placement of the earth backing (Photo 179). Earth fill was placed along the river and landside of the bottom plank and compacted to fill the plowed furrow. The remainder of the earth backing consisting of a mixture of gravel, sand, silt, and clay, was uncompacted.

57. Section 4-4 was exposed to static differential heads of 1.0, 2.0, and 2.5 ft for periods of 24 hr each, and no measurable seepage was noted at any time. During this period, the section was exposed to 3 to 5 in. of rain which saturated the earth backing. By the end of the 2.5-ft static differential head test, the crown of the earth backing had consolidated about 2 in., and the flashboard had developed a 2- to 3-deg lean toward the riverside in some areas (Photos 180 and 181). Following the static head tests, the water level was lowered to 1.0 ft, and the structure was exposed to 19 hr of wave attack over a period of 192 hr (Photo 182). Saturation of the earth backing by heavy rains and the pumping action of the waves resulted in an increase in the riverward lean of the flashboard and additional consolidation of the earth fill. The top of the flashboard had leaned as much as 4 in. riverward in some areas. Some areas of the earth fill crown were lowered as much as 8 in. No obvious seepage was noted, and the structure was in good condition at the end of wave attack at the 1.0-ft water level (Photos 183 and 184).

58. The water level was raised to 2.0 ft, and the structure was exposed to 8.5 hr of wave attack over a period of 48 hr (Photo 185). Some sporadic wave overtopping occurred during this test condition. The amount of overtopping water was sufficient to keep the crown of the earth backing saturated but was not enough to produce runoff and surface erosion. By the end of this test condition, the flashboard showed some additional riverward lean and earth

crown lowering. The test section was in good condition and did not appear to be accruing any additional damage, and no measurable seepage was observed (Photos 186 and 187). Wave attack at the 2.5-ft water level produced severe and continuous overtopping (Photo 188). The overtopping water produced very rapid erosion of the earth backing (Photos 189-192). After approximately 2.5 hr of wave attack, the structure showed major deterioration. The 6-mil poly backing had torn loose from the flashboard, and the crown and slope of the earth backing showed major erosion. When the poly tore loose, it was noted that overlap joints showed minor seepage; but scab joints showed no seepage. The pump was kept running in order to replace water being lost from the small basin by wave overtopping.

59. When the wave machine was stopped, water loss from the small basin also stopped, indicating no major seepage or boils had formed. The flashboard showed continuous flexure with each impinging wave, and this movement increased as the earth backing was lost (Photo 193). After 3.0 hr of wave attack, a boil formed (Photo 194), and water loss from the small basin became so rapid that even with the wave machine turned off (wave overtopping stopped) the pump could not maintain the 2.5-ft water level. The pump was turned off, and the small basin was allowed to drain through the failed area of the test section. Photos 195-198 show the condition of Section 4-4 at the end of the entire test.

4-ft-high plywood mud box with earth fill

60. CELMK (1975) recommends a plywood mud box with earth fill when the required levee capping height exceeds 3.0 ft. Construction guidelines call for removal of grass and blading of the existing ground to provide a uniform grade prior to placing mud boxes. The riverside and landside panels can be prefabricated and stored until needed during emergency operations. During field assembly, 4-ft-long, 4- by 4-in. posts are to be placed a minimum of 2 ft into the existing levee at a distance of 5.25 ft from the riverside of the levee crown. The panels are tied together with wire and 2- by 4-in. braces, and the completed 8-ft section of mud box is placed against the 4- by 4-in. posts. The adjacent 8-ft sections of mud boxes are nailed together, and a 6-in.-wide scab is nailed over the lower portion of the riverside joint between adjacent boxes. The boxes are then filled to a 2.0-ft height with tamped earth fill.

61. Section 4-5 (Plate 27 and Photos 199 and 200) was a 4-ft-high plywood mud box with earth fill. The section was constructed following the guideline described in the preceding paragraph. After discussions with CELMK, the 6-in.-wide scab joint between adjacent mud boxes (Item "G," Plate 27) was modified to extend the full 4-ft-height of the butt joint. The scab was sealed with plastic roofing cement and nailed to adjacent mud boxes. The earth fill was placed in lifts and lightly tamped by personnel walking over the fill. CELMK felt that this would closely simulate the fill compaction that could be achieved by conventional construction methods. Photos 201 and 202 show Section 4-5 prior to and during placement of the tamped earth fill. The earth fill was visually described as a gravelly, sandy, clayey silt.

62. While the small basin at the start of the 1.0-ft static differential head was being filled, seepage became quite noticeable when a 0.3-ft static differential head was reached. Seepage became quite significant, and water was being lost underneath the structure at about the same rate it was being pumped into the small basin (approximately 700 gal/min). Photo 203 shows rills that were cut in the existing levee soil on the landside of the structure by water flowing under Section 4-5. In an effort to slow down the seepage, a row of sandbags was placed along the riverside toe of the test section (Photo 204). However, the sandbags had little or no effect on the seepage rate observed during the 1.0-ft static differential head. Heavy rains occurred during the next two days, and the structure stood with no static differential head. The earth fill became saturated and consolidated in several areas (Photo 205).

63. In an effort to compact the fill, personnel walked over and tamped the earth fill. It was evident during this tamping that flow under the structure had cut caverns in the fill material. After compacting the fill as much as feasible by walking over the surface of the earth (Photo 206), the structure once again was exposed to a 1.0-ft static differential head. The structure showed only minor seepage, and the small basin dropped only 0.2 ft (equivalent to a 0.8-ft static differential head) over a period of 24 hr. The mud boxes were filled to their original 2.0-ft height with tamped earth fill (Photo 207) and then exposed to 24 hr of 2.0-ft static differential head. The fill material became saturated, and moderate amounts of seepage occurred along the full length of the riverside toe which resulted in the loss of 0.5 ft of

water from the small basin in 24 hr. Photo 208 shows Section 4-5 after 24 hr of exposure to the 2.0-ft static differential head.

64. The static differential head was then raised to 2.5 ft, and seepage under the structure became quite significant. Photo 209 shows the obvious free flow of water that was occurring under the structure. The pump was left running in order to maintain the 2.5-ft differential head. At the end of the workday the pump was turned off. By the following morning the small basin had drained, and the structure showed severe undermining and consolidation of the earth fill (Photo 210). The earth fill was once again repaired, with fill placed and tamped along landside toe (Photo 211), and the structure was exposed to 3.5 hr of wave action at the 1.0-ft water level. The structure performed very well with only very minor amounts of seepage. The water level was raised to 2.0 ft, and the structure was exposed to 1.0 hr of wave action. There was a noticeable increase in seepage but in only a moderate amount. The water level was left at 2.0 ft, and at some time over the weekend the structure failed. A large area of the fill undermined and caved in, and the small basin drained to a 0.0-ft differential head level (Photos 212 and 213).

4-ft-high planking mud box with tamped earth fill

65. The US Army Engineer Districts, Louisville (CEORL) (1975) and Seattle (CENPS) (undated) recommend a planking mud box as one alternative to topping an existing levee. CENPS was not as specific as CEORL in its construction and usage guidance for this type of structure. Therefore, CEORL guidance was followed for construction of this test section. The planking mud box structure is recommended by the CEORL for use when topping is to be equal to or greater than 3.0 ft in height or is to be placed on a narrow crown levee. The width of the box should be equal to the height of the earth fill, but at no time should it be less than 3.0 ft. Support posts should be spaced approximately 3 ft apart and driven 2.0 to 3.0 ft into the existing levee. The bottom row of boards on both the riverside and landside of the structure should be butted end to end, with these joints falling at the support posts. Joints on the upper rows can be staggered. The bottom row of boards should extend into the existing levee, thus providing a seepage key and bonding trench. The landside of the riverside planking should be covered with a single layer of sacking material prior to placement of well tamped earth fill.

Upper ends of the support posts are to be held together with number 9 form wire.

66. Section 4-6. Section 4-6 (Plate 28 and Photos 214 and 215) was a 4-ft-high planking mud box with tamped earth fill. The test section was constructed following the guidelines described in the preceding paragraph. Photos 216-218 show Section 4-6 during various construction phases. The structure was exposed to static differential heads of 1.0, 2.0, 3.0, and 3.5 ft for periods of 24, 72, 24, and 24 hr, respectively. The structure showed no instability or seepage for any of the test conditions. Photos 219 and 220 show the test section at the end of the 3.5-ft static differential head. Following the static differential head tests, the small basin water level was lowered to a 1.0-ft depth, and the structure was exposed to 19 hr of wave action over a period of 168 hr. The test section then was exposed to 4.5 hr of wave action at the 2.0-ft water level over a period of 48 hr. During these two wave and swl conditions, the structure sustained no damage. Wave action at the 3.0-ft water level produced moderate to significant overtopping which kept water ponded on the earth fill (Photos 221 and 222). After 9 hr of wave action at the 3.0-ft level over a period of 96 hr, the earth fill had been lost in one area. With the water ponded on top of the fill, piping of water down through the earth fill and out the landside toe of the structure had caused this material loss. The structure could have been repaired quite easily, but this condition showed that the structure is susceptible to damage once a wave and swl combination is reached that causes wave overtopping (Photos 223-225).

67. Section 4-6-A. Section 4-6 was a very rigid structure with support posts being placed on 3.0-ft centers. It was a very labor-intensive structure to build. In an effort to reduce construction requirements and reduce cost, Section 4-6-A was tested to see what effect the removal of every other support post would have on overall stability of the test section. Tie wires were removed, support posts were cut off at ground level, and the earth fill was repaired (Plate 28 and Photos 226 and 227) prior to exposing the structure to static differential heads of 3.0 and 3.5 ft. Each of the static head conditions were maintained for 24 hr, and the structure showed only minor seepage at the 3.5-ft level (Photo 228). The water level was lowered to 3.0 ft, and the structure was exposed to 6.5 hr of wave action over a period of 24 hr. In the early stages of this test, damage to Section 4-6-A proceeded much like it

had on Section 4-6 when exposed to overtopping wave conditions. Water ponded on top of the earth fill and began to pipe down through the earth and out the landside toe. As earth fill began to leach out of the structure toe, it was noted that riverside planking began to flex with incident wave attack, and landside planking began to bulge out where the support posts had been removed. This occurrence resulted in a free flow of water and suspended earth fill through gaps between the flexing boards (Photo 229). Thus, it could be seen that increasing spacing between support posts from 3.0 to 6.0 ft had an adverse effect on the structure's stability when exposed to moderate overtopping wave conditions. Photo 230 shows the condition of Section 4-6-A at the end of the test.

Six-Foot-High Structures

Plastic grid with sand fill

68. Section 6-1 (Plate 29 and Photos 231-233) was a 6-ft-high structure constructed of nine lifts of plastic grid filled with masonry sand. For this test, GL at CEWES supplied a medium strength black plastic grid which was cut to the lengths and widths needed to construct Section 6-1. Except for use of a spun woven filter fabric between successive lifts and slight compaction of sand in the bottom lift to produce a better foundation, construction techniques used on this section were identical to those described for Section 4-2 in paragraphs 36-38. Photos 234-237 show Section 6-1 at various stages of completion. This section was constructed during warm weather, and the plastic grid had a thinner wall thickness than the grid used on Section 4-2. Therefore, no major problems were encountered during expanding and placing the grid. While the use of filter fabric between lifts eliminated the problem of sand loss due to misalignment of cells in successive lifts, efforts still were made to maintain good cell alignment. Due to the increase in height, Section 6-1 was constructed two cells wider than Section 4-2.

69. The structure was exposed to 24 hr each of 1.0-, 2.0-, 3.0-, 4.0-, and 5.0-ft static differential heads. The structure performed quite well for all these conditions, but seepage through the structure, even though it did not become significant, increased with increasing static differential head. It appeared that the filter fabric had a capillary action which pulled water through the structure. It also is felt that as the sand became wet and

settled in the cell, a void area was produced between the sand and filter fabric which in turn produced a lower resistance route for the water to move through the structure. Between the 3.0- and 4.0-ft static differential head tests, ring levees were constructed at the ends of the structure on the riverside. This was done to stop the flow of water around the ends of the structure. A combination of cell breakage and leaching of sand out of the bottom resulted in a minor loss of sand from the first row of cells in each lift on the riverside of the structure. This loss of sand and a minor bow in the structure, which occurred during the 5.0-ft static differential head test, were the only damage noted at the end of the static head tests (Photos 238-241).

70. The basin was flooded back to a 3.0-ft depth, and the structure was exposed to 18 hr of wave action over a period of 168 hr. After 5 hr of wave action, the basin was drained in order to inspect the riverside of the structure. Several of the cells in the first row of lifts 4, 5, and 6 on the riverside of the structure had started to split open and had lost the majority of their sand fill (Photo 242). After 18 hr of wave action, all damage appeared to have stopped. After the basin was drained, a close inspection of the structure revealed that many of the cells in the riverside row of lifts 2 through 6 had partially or completely split open and had lost their sand fill (Photo 243). Cell breakage occurred adjacent to spot welds, indicating a weakness in the plastic itself and not the welds. Sand had accumulated at the structure toe and had nearly buried the first lift of cells (Photo 244). Only sporadic cell breakage was noted on the landside, and a slight increase in the bow in the structure was noted at the end of wave action at the 3.0-ft water level (Photos 245-247). The basin was flooded to the 4.0-ft water level and the structure exposed to 19 hr of wave action over 196 hr. After approximately 18 hr of wave action, all cells in the first two rows on the riverside and a majority of cells on the landside row had split open and emptied. The structure had developed a very prominent bow in the middle and a landside lean. Seepage through the structure was still minor, but it had increased from the amount observed at the 3.0-ft level. Except for an occasional cell splitting on the landside of the structure and some additional emptying of a few riverside cells, no additional damage was noted during the last 1 hr of wave action. Photos 248-251 show the condition of Section 6-1 at the end of wave action at the 4.0-ft water level. The structure was showing some obvious

loss of structural integrity, but it was still functioning adequately.

71. During flooding of the basin to carry out wave action at the 5.0-ft water level, the structure failed. When the water had reached 4.9 ft, the structure was displaced landward by a combination of overturning and sliding (Photo 252). Sliding occurred along the top surface of the second lift of cells. Photos 253 and 254 show Section 6-1 after failure.

Planking mud box
with tamped earth fill

72. Section 6-2 (Plate 30 and Photos 255-257) was a 5-ft-high planking mud box with tamped earth fill. Except for its additional height and width, Section 6-2 was identical in construction to Section 4-6 described in paragraph 65. The 5-ft height is the maximum height recommended in CEORL's and CENPS's emergency manuals. Photos 258-261 show Section 6-2 at various stages of construction.

73. The structure was exposed to 1.0-, 2.0-, and 3.0-ft static differential heads for 24 hr each and 4.0- and 4.5-ft static differential heads for 96 and 48 hr, respectively. Heavy rains occurred during this time which added to the saturation and consolidation of the clayey silt fill material. At the completion of the 4.5-ft static head test, fill material had settled from 5 to 8 in., but the structure showed no obvious seepage. Photos 262-264 show the condition of Section 6-2 after the basin was drained at the conclusion of the static differential head tests. Comparison of Photos 256 and 263 shows that the planking and support posts shifted slightly during these tests, but this did not appear to affect structural stability of the mud box.

74. The basin was filled to the 3.0-ft water level, and the section was exposed to 6.5 hr of wave action over a period of 96 hr. This wave condition produced no overtopping. No seepage was observed, and the structure sustained no damage. The water level was increased to the 4.0-ft level. A total of 8.5 hr of wave action over a period of 72 hr caused significant damage to the earth fill. Significant, but sporadic, wave overtopping (Photo 265) resulted in water pooling on the earth fill. This standing water began piping down along the landside of the fill and exited the structure around the bases of two support posts (Photo 266). This flow of water caused the continuous slow removal of fill material which eventually would have failed the structure for this condition. Photo 267 shows the deteriorated condition of the earth fill

when the tests were stopped, while Photos 268 and 269 show that the planking and support posts were still in good condition.

6-ft-high plywood
mud box with earth fill

75. CELMK (1978) recommends a 6-ft-high plywood mud box with earth fill when the required levee capping height exceeds 4.0 ft. Except for panel heights and spacing, recommended construction of this section is identical to the 4-ft-high plywood mud box, Section 4-5 (Plate 27), described in paragraphs 60 and 61.

76. Section 6-3. Section 6-3 (Plate 31 and Photos 270 and 271) was a 6-ft-high plywood mud box with tamped earth fill. The earth fill had a visual classification of clayey silt. Because of the problem of seepage under Section 4-5, CELMK requested that we entrench the 6-ft-high riverside panel 2 in. into the existing levee. CELMK stated that they could cut a slit trench and drive the panel into it to obtain a tight seal; however, if this were not possible and a wider trench were cut, it was recommended that the panel be placed against the riverside edge of the trench and the trench fill material be compacted against the landside of the panel. As in Section 4-5, a 6-ft-high, 6-in.-wide scab was used to connect the butt joints on the riverside of adjacent 8-ft-mud box sections. The scab was sealed with plastic roofing cement and nailed on 6-in. spacings. The earth fill was placed in lifts and was lightly tamped by personnel walking over it.

77. The structure was exposed to 24 hr of 1.0-ft static differential head (Photos 272 and 273). During this time, 2 in. of rain fell, and the landside of the structure was saturated. For this reason, it was hard to see if any minor seepage was occurring, but there was obvious seepage around one of the landside support posts. The seepage rate did not seem to increase with time, and for this reason the static differential head was raised to 2.0 ft. After 1.0 hr, the structure failed. The area around the support post which had exhibited obvious seepage during the 1.0-ft differential head was the failure point. After the small basin was drained, it was obvious where water had undermined the 6-ft-high riverside panel, passed under the structure, and exited in a boil around the landside support post. The tunneling action of flow under the structure removed a large amount of fill material.
Photos 274-276 show the condition of Section 6-3 after failure at the 2.0-ft static differential head.

78. <u>Section 6-3-A.</u> After discussions with CELMK, it was decided that the 6-ft-high panel needed to be entrenched deeper into the existing levee to determine if this would prevent water from undermining the structure. It was decided that the area of failure on Section 6-3 would be repaired and a 4-in. berm would be constructed on the riverside of the structure which would simulate a total 6-in. entrenching of the 6-ft panel. The failed fill area was cut out (Photo 277), and the area of undermining was filled and compacted well to try to achieve an existing levee compaction density. The fill then was replaced and tamped in the same manner as the original construction. A 4-in.-high, 18-in.-wide berm of material was constructed on the riverside of the 6-ft-high panel. This berm was well compacted and constructed of existing levee soil. The repaired section was referred to as Section 6-3-A (Plate 32 and Photos 278 and 279). Heavy rains occurring after repair was completed and before testing of Section 6-3-A was initiated accounted for the wet appearance in before-test photographs.

79. Except for seepage around the ends of the structure, Section 6-3-A performed very well for static differential heads of 2.0, 3.0, and 4.0 ft. After 1.0 hr at the 5.0-ft static differential head, the structure failed when boils formed on the landside of the structure and at several points in the earth fill (Photo 280). Seepage around the ends of the structure kept the earth fill saturated, and this probably weakened and eventually resulted in failure of the earth fill to withstand the pressures developed by the 5.0-ft static differential head.

80. <u>Section 6-4.</u> After discussions of the test results of Section 6-3-A with CELMK, it was decided to make design modifications in the 6.0-ft-high plywood mud box that would allow for its 6-in. entrenchment into the existing levee. This new design was referred to as Section 6-4 (Plate 33 and Photos 281 and 282). The vertical 2-in. by 4-in. braces on the riverside panels (item "K", Plate 33) where shortened to 5 ft 6 in. so they would not extend into the trench. A 6-in.-deep trench was excavated, and the riverside panel (Panel B) was placed as far riverward in the trench as possible (Photo 283). The clayey silt fill material was thoroughly compacted in the trench with hand tampers, while the remainder of the clayey silt earth fill was dumped from a small bucket loader (Photo 284), distributed with shovels, and tamped into place by personnel walking over the fill that was placed in 6- to 8-in. lifts.

81. Section 6-4 was exposed to 48 hr of 1.0-ft static differential head. During this time, the structure showed minor seepage under the structure, but the seepage was clear which indicated that no fill material was being eroded (Photo 285). The static differential head was increased to 2.0 ft and maintained at this level for 24 hr. With the increase in static head, there was a noticeable increase in seepage, but the amount of seepage was still minor, and the seepage water remained clear (Photo 286). The seepage rate showed a significant increase when the static differential head was increased to 3.0 ft. The seepage waters became quite murky, indicating that erosion of fill material was occurring. After 4 hr at the 3.0-ft head, seepage boils began to form around some of the vertical supports on the riverside panel. The severity of these boils and the rate of fill erosion increased quite rapidly. In 20 min, the water level had dropped 1 ft. The rate of erosion and severity of the boils decreased as the water level dropped, but it was obvious that the structure had failed. Damage would have been more severe if the static head had been maintained. Photo 287 shows the eroded earth fill after the remaining water was pumped from the small basin and the structure was allowed to dry for approximately one day. Close inspection of the structure revealed that compacted earth in the riverside trench had failed and most likely initiated the failure of the earth fill. Photo 288 shows a portion of the eroded trench area below the lower horizontal support on the riverside of Panel B.

82. During construction of Sections 6-3, 6-3-A, and 6-4, it was noted that the horizontal braces on the riverside of Panel B made it very difficult, if not impossible, to obtain a thorough compaction of the fill in the portion of the trench on the riverside of the structure. Also, the proximity of the 2-in. by 4-in. bracing to the existing levee on the inside of the mud box, along with the quantity of bracing material, made it very difficult to fill the mud boxes and not leave void areas underneath and adjacent to these braces.

83. Section 6-5. Weakness of the earth fill in the trench and possible voids left in the fill material were probable causes for the ultimate failure of Section 6-4. After discussions with CELMK, it was decided to further modify design of the 6-ft-high plywood mud box. This design was referred to as Section 6-5 (Plate 34). The horizontal braces were removed from the riverside of Panel B. Facing the riverside of the mud box, the horizontal and

diagonal braces (items "G" and "E," respectively) were removed from the left-hand side of each mud box, and the remaining horizontal braces (item "G") were raised 1.0 ft. The wire ties were moved inward to the nearest vertical braces. These modifications would allow the use of vibrating and/or hand tampers to compact the fill in the trench on the riverside of the mud boxes and would improve the quality of earth fill placement in the mud boxes.

84. The mud boxes were modified as described above, set in place with the riverside plywood positioned in the middle of the 6-in.-deep trench and nailed together (Photo 289). The riverside of the trench was backfilled with clayey silt and thoroughly compacted by hand, while the landside of the trench (landside of Panel B) was filled and compacted by foot tamping. As was done on Section 6-4, the remaining earth fill (clayey silt) was placed with a small bucket loader, distributed by shovel in 6- to 8-in. lifts, and lightly tamped by walking (Photo 290). Photos 291 and 292 show Section 6-5 after construction was completed and before tests were started.

85. Section 6-5 was exposed to 1.0-, 2.0- and 3.0-ft static differential heads for 24 hr each. The 1.0-ft head produced no seepage, while a small amount of seepage was noticeable after 2 hr of 2.0-ft head. This seepage stopped, however, after 4 hr. A minor amount of sustained clear water seepage was observed throughout the 3.0-ft head test. After 24 hr at the 3.0-ft head, about one half of the fill was wet, and a small amount of water had ponded around two of the vertical supports on Panel B (Photo 293). No increase in seepage rate was noted after 24 hr, and no obvious erosion of the earth fill was occurring. The static head was raised to 4.0 ft. After 2 hr, the entire earth fill was wet, and water had pooled in several areas on the earth fill (Photo 294). The water appeared to be coming up the riverside of the earth fill adjacent to the vertical supports, and plywood and was moving across the fill and piping down the landside of the fill adjacent to the plywood and vertical supports of Panel A.

86. After 24 hr, the seepage rate had not increased, but the static condition was maintained for an additional 72 hr to see if the structure would fail or sustain any appreciable damage. After 96 hr of 4.0-ft head, water had ponded in several areas of the saturated earth fill, which showed some consolidation and minor surface erosion (Photo 295). The seepage rate had not increased over the 96 hr, and the water remained clear. When the static head was increased to 5.0 ft, the seepage rate increased but was still only

moderate in severity; and the water was clear. The water level dropped 0.6 ft over a 24-hr period, showing that the seepage rate had doubled over that observed at the 4.0-ft level. A noticeable bow had formed near the center of the 24-ft span of mud boxes. The support posts showed a landward lean, but the structure showed no evidence of being at a point of structural failure (Photos 296-298).

87. Water ponding on the earth fill had eroded around the end of Panel A where it was in contact with the bank of the cut area in the existing levee. This ponding caused some minor erosion of the earth fill but did not show any signs of causing a failure of the earth fill (Photo 298). This damage was not felt to be representative, except for a prototype condition where the mud box was being used to close a cut area. In that case, sandbags or some other type of reinforcement could be used in this area. The mud boxes were exposed to 5.5 hr of wave action at the 4-ft water level over a period of 168 hr. This combined wave and swl caused only minor, sporadic wave overtopping (Photos 299 and 300). The waves impacting on Panel B caused some very slight flexing of the plywood, but this did not seem to increase in severity or result in additional problems. The wave overtopping did result in an increase in rate of water loss from the small wave basin and increased ponded water on the earth fill. The seepage rate through the structure showed no noticeable increase when wave action was stopped, and the earth fill showed only superficial surface erosion. The water level was increased to 4.7 ft, and wave action was continued to see what effects increased loading would have on the structure and increased overtopping would have on the earth fill. This condition caused significant, sporadic wave overtopping, but because of the amount of clay in the fill, the erosion rate was quite slow. Panel B showed a slightly higher degree of flexing, but it showed no signs of failure. This condition was maintained for 1.5 hr, and the only damage noted was a small trench cut in the earth fill where overtopping water was impacting. (In Photo 301 note ponded water in trench cut by overtopping water.) The water level was increased to 5.5 ft (top of Panel B) and left overnight. This was done to see if the structure, in its somewhat deteriorated condition, could withstand this maximum static load. The seepage rate showed a slight increase (1.5 ft of water lost overnight), but the structure stood the test condition quite well. Photos 302-304 show the condition of the structure after the basin was drained.

PART IV: CONCLUSIONS

88. Based on the test conditions, durations, and results reported herein, the following conclusions were reached:

a. Sections 2-1, 2-2, 2-3, 2-4, and 2-5 are equally adequate 2-ft-high expedient levee-raising structures for static differential heads up to and including 1.5 ft, allowing only minor amounts of seepage.

b. Sections 2-1, 2-2, 2-3, and 2-4 are not adequate 2-ft-high expedient levee-raising structures for placement in wave action environments.

c. If the fill material is coarse enough to prevent leaching of material out of the sacks, Section 2-5 would be an adequate 2-ft-high expedient levee-raising structure for placement in mild wave action environments (wave heights less than 0.5 ft).

d. The clayey silt-filled, spun woven and woven polypropylene sacks and the clay gravel-filled burlap sacks are equally adequate building elements for 4-ft-high earth-filled sack structures exposed to static differential heads up to and including 3.35 ft, provided they are assembled in the manner described for Section 4-1.

e. The three types of sacks with their respective fill materials, when assembled in the manner described for Section 4-1, are adequate to use in the construction of 4-ft-high earth-filled sack structures which will be exposed to mild wave action (wave height less than 0.75 ft) for lengths of time that do not greatly exceed the tested durations. The degree of damage which the structures will sustain when exposed to wave action is dependent upon the following:

   (1) Size of waves, water level, and duration of wave attack.

   (2) Degree of quality control obtained during construction of the structure.

   (3) Type of sacks and fill material being used and the length of time that the structure is in service.

   The woven polypropylene sacks have a very slick texture which makes it difficult to get a good folded seal where the sacks overlap, and this results in leaching of the fine fill material out of the end folds. The spun woven polypropylene and burlap sacks show progressive deterioration with extended exposure to sunlight, rain, and other weathering elements. This deterioration in turn results in loss of fill material and deterioration of the structure.

f. The 4-ft-high sand-filled plastic grid, Section 4-2, both with and without filter cloth or burlap between lifts, is an adequate design for static differential heads up to and including 3.0 ft. The sections with burlap and filter fabric sustained

less damage and showed slightly less seepage than the unfiltered section of the structure.

g. The unfiltered portion of Section 4-2 is not an adequate design for exposure to wave attack. Cell breakage in this section produced a more rapid failure than would have occurred had the cells not broken, but the structure would have eventually failed due to cell emptying induced by pumping action of the waves.

h. The portions of Section 4-2 which contain either filter fabric or burlap between successive lifts appear to be adequate designs for placement in mild wave environments (wave height less than 0.75 ft). Due to the failure of the center section, it is unknown how long these sections could hold up under wave attack. Based on the limited wave attack that they were exposed to, and if care is taken in selecting a plastic grid that will not deteriorate and break, the sand-filled plastic grid with filtering should perform as well as the earth-filled sacks, Section 4-1.

i. Section 4-3, plywood flashboard, will show immediate seepage (removing fill material from the trench) when exposed to a 1.0-ft-static differential head; and the structure will fail under this static differential head in less than 24 hr.

j. Section 4-3-A, plywood flashboard with 2-ft-high compacted clayey silt backing, is an adequate design and will have little or no seepage for static differential heads up to and including 2.9 ft.

k. Section 4-3-A is a very adequate design for exposure to mild wave climates (wave height less than 0.75 ft) for water depths up to and including 2.0 ft. For mild wave attack at water levels equal to or exceeding 2.5 ft, the earth backing will require occasional maintenance due to erosion caused by overtopping waves. The frequency and degree of maintenance will depend on the degree of overtopping and the erodibility of the earth backing. If maintenance is not carried out, the section will eventually fail if exposed to continuous wave overtopping conditions.

l. Section 4-3-B, plywood flashboard with 1-ft-high compacted clayey-silt backing, is a very adequate design for static differential heads up to and including 1.5 ft. Some seepage and damage to the earth backing will occur for static differential heads greater than 1.5 ft but less than or equal to 2.5 ft. The structure will most likely fail if the static differential head exceeds 2.5 ft.

m. Section 4-3-C, plywood flashboard with 2-ft-high uncompacted clayey-silt backing, is a very adequate design for static differential heads up to and including 2.0 ft. Some minor seepage and erosion of the earth backing will most likely occur for static differential heads greater than 2.0 ft but less than or equal to 2.5 ft. Static differential heads greater than 2.5 ft will most likely result in failure of the structure.

n. Section 4-3-D, plywood flashboard with asphalt sealer, is an adequate design for static differential heads up to and including 2.0 ft provided the asphalt has adequate time to cure prior to being exposed to any static differential head condition. Static differential heads exceeding 2.0 ft will most likely result in failure of the structure.

o. Due to their failure during the static differential head tests, Sections 4-3-B, 4-3-C, and 4-3-D were not tested under wave attack. Based on observations during their static head tests and the results of the wave tests on Section 4-3-A, it can be stated with some degree of confidence that Sections 4-3-B and 4-3-C should be adequate designs for placement in mild wave climates as long as the still-water depths do not exceed the static differential head levels which the structures were found to be adequate for and the wave climate does not produce any significant wave overtopping. Section 4-3-D should be adequate for placement in mild wave climates as long as the still-water depth does not exceed 1.5 ft.

p. Section 4-4, 3-ft-high planking flashboard with earth backing, is a very adequate design for static differential heads up to and including 2.5 ft. Both the 6-mil polypropylene and the sacking material provided adequate protection to prevent leaching of the earth backing; therefore, one cannot be judged better than the other.

q. Section 4-4 is an adequate design for placing in areas exposed to mild wave climates as long as the combined water depth and wave conditions do not produce wave overtopping. Wave overtopping will result in erosion of the earth backing. The erosion rate and thus the amount of time it would take for a structure to fail will be dependent upon the amount and frequency of wave overtopping and the erodibility of the earth backing.

r. Section 4-5, 4-ft-high plywood mud box with earth fill, may or may not be an adequate design for static differential heads equal to or less than 1.0 ft. The adequacy of the design is highly dependent upon how well the earth fill is compacted. If extra care is taken to adequately tamp the fill, the structure may be an adequate design for very short durations of static differential heads up to and including 2.0 ft.

s. Section 4-5 is an adequate design for placement in mild wave climates, provided the fill is well tamped and the static differential head does not exceed 1.0 ft.

t. Section 4-6, 4-ft-high planking mud box with tamped earth fill, is an adequate design for static differential heads up to and including 3.5 ft.

u. Section 4-6 will sustain some slow progressing damage (erosion of earth fill) when placed in a combined wave height and swl condition that produces wave overtopping. The structure is a very adequate design for placement in mild wave climates that do not produce wave overtopping.

v. When the support post spacing on Section 4-6 is increased from 3.0 to 6.0 ft (referred to as Section 4-6-A) the structure is still an adequate design for static differential heads up to and including 3.5 ft, but Section 4-6-A is likely to accrue damage more rapidly than Section 4-6 when exposed to overtopping wave conditions.

w. Section 6-1, 6-ft-high plastic grid with sand fill and spun woven filter fabric between lifts, appears to be an adequate design for static differential heads up to and including 4.0 ft. The structure should contain static heads up to 5.0 ft, but some landward slippage of the structure could occur for static differential heads exceeding 4.0 ft.

x. Section 6-1 appears to be an adequate design for static heads but will sustain minor to moderate damage when placed in a mild wave environment (wave heights not exceeding 0.75 ft) with swls not exceeding 3.0 ft. A combination of mild wave attack with swls exceeding 3.0 ft will result in higher degrees of damage that could ultimately lead to failure of the structure.

y. Section 6-2, 5-ft-high planking mud box with tamped earth fill, is a very adequate design for static differential heads up to and including 4.5 ft.

z. Section 6-2 is an adequate design for placement in a mild wave climate as long as the combined wave and swl condition does not produce wave overtopping. The rate of damage sustained by the structure due to wave overtopping appears to be directly proportional to the degree of overtopping. Thus, in a wave overtopping environment the structure's useable lifetime is dependent upon the amount of wave overtopping.

aa. Section 6-3, 6-ft-high, 2-in. entrenched plywood mud box with earth fill, is a marginally acceptable design for static differential heads up to 1.0 ft. Static differential heads exceeding this will most likely result in undermining and eventual failure of the structure.

bb. Section 6-3-A is not a recommended design. It was only tested to see if there were any need to redesign and test a 6-ft-high plywood mud box with a 6-in. entrenchment of the 6-ft-high riverside panel.

cc. Section 6-4, 6-ft-high, 6-in. entrenched plywood mud box with earth fill, is an acceptable design for static differential heads up to and including 2.0 ft. The stability of the structure is highly dependent upon the type of earth fill and its degree of compaction. For a clayey silt fill, which was placed in a dry condition and tamped in place by personnel walking on the material, a static differential head greater than 2.0 ft will most likely result in failure of the structure's earth fill.

dd. Section 6-5, modified 6-ft-high, 6-in. entrenched plywood mud box with earth fill, is a very adequate design for static

differential heads less than or equal to 3.0 ft. The structure is an adequate design but may exhibit some minor seepage and sustain some minor damage to the earth fill and deformation of the wooden superstructure when exposed to static heads greater than 3.0 ft but less than or equal to 5.0 ft.

ee. Section 6-5 is a very adequate design for placement in mild wave climates as long as the combined wave and swl conditions do not produce significant amounts of wave overtopping. The degree of damage sustained by the earth fill due to wave overtopping will be dependent upon the amount of overtopping and the type of earth fill material used and its degree of compaction.

PART V: DISCUSSIONS AND CONSIDERATIONS

89. The tests reported herein did not address the stability problems related to flow velocities along the structure, debris in the water, scour and undermining of the structure toes, and high amplitude wave action. The scour caused by flow velocities and debris hitting the various structures could have substantial effects on their ability to provide protection against static differential heads and could lead to more rapid erosion in a wave action environment. Polyethylene covers or wraps could be destroyed by floating debris and by flow velocities pulling them away from the structures. The earth-filled sacks and sand-filled plastic grids with filters between lifts appear to provide adequate protection in a low amplitude wave environment. Though not tested, debris impacts could cause considerable damage to the sacks and grids and/or cause the structure to fail. The plywood and planking flashboards and mud boxes should be less susceptible to damage by debris.

90. When water levels and wave action were conducive to scour and/or undermining a structure's toe, the stability of the structure could be substantially reduced. Therefore, if wave action is expected at a structure's toe, it is recommended that some type of erosion protection be placed in this area and on the face of the existing levee.

91. Highly variable types of soil might be used to construct expedient structures throughout the United States. Due to the limited scope of this study, only two locally available but widely different types of soil (clayey silt and gravelly clayey sand) were used in this test series. In general, depending on the grain size distributions and cohesiveness of the soils used, the widths of a potato ridge, earth backing behind a flashboard, and mud box needed to prevent piping and boils will vary. Obviously, very sandy soils would require a wider structure than would be needed with cohesive clays.

92. For earth-filled sack structures placed in wave action environments, care must be taken to select a sack and fill material that match. The more porous the sack, the coarser the fill material needed. If the fill material has sharp edges, such as crushed shells or stone, care must be taken to select sacks constructed of stronger, more tear-resistant materials.

93. The 3-ft-high planking flashboard with earth backing, Section 4-4, had a tendency to develop a riverward lean which potentially could have resulted in a more rapid failure of the structure. Where high static

differential heads and/or moderate wave action are expected, some type of bracing on the landside of the structure could be used to prevent its riverward lean. This may add to overall stability of the structure and result in a structure that would sustain damage at a somewhat slower rate.

94. Whether support posts are driven or placed in predug holes, care must be taken to tamp fill back into any open areas around the posts. If this is not done, seepage could initiate at these areas which could result in the failure of a structure that was otherwise well built.

95. When synthetic materials (plastic, polypropylene, sacking material, etc.) are used on a structure, care should be taken to select materials that will not show rapid deterioration under ultraviolet light. The cell breakage on Sections 4-2 and 6-1 (sand-filled plastic grid) was due to an inherent weakness in the plastic combined with deterioration that resulted from weathering. The performance of the sections would have been much better if cell breakage had not occurred.

96. Based on results of the study reported herein, Section 6-1 would be an improved design if the filter fabric placed between lifts did not extend continuously from riverside to landside. Instead, the filter fabric should be placed two cells deep on the riverside and landside and with no filter between lifts on the center cells. In this way, as the sand becomes wet and settles, the lower cells would be fed sand from the upper cells, and no gap would occur for the free flow of water through the structure. Placement of the filter fabric in the manner mentioned above also should give some small amount of additional bond strength between lifts and help prevent landward sliding of the lifts. Additional testing is needed to better understand and improve the design of the sand-filled plastic grid structure.

97. The planking mud boxes with tamped earth fill would have performed better when exposed to wave overtopping if the mud boxes had been completely full of fill material. If this had been the case, the water could have run off more readily instead of ponding, piping down through the structure, and exiting around the landside support posts. Thus, if a mud box is exposed to wave overtopping, it is conjectured that its useful life can be extended if the boxes are kept full of earth fill.

98. The performance of the plywood mud box with earth fill was greatly improved by removing horizontal bracing from the riverside plywood panel which allowed room for tampers to be used to compact the fill material placed in the

6-in.-deep riverside trench. This modification, along with raising the internal bracing to improve placement of a uniform tamped earth fill, resulted in a highly improved structure relative to the original design. It must be noted that success of this modified design is highly dependent upon achieving good compaction of the fill in the riverside trench and obtaining a uniformly tamped earth fill that is devoid of air pockets and lines of weakness that could serve as low resistance piping points for seepage water. The landward lean of this structure, when exposed to high static differential heads, shows that the use of support posts on the landside of the structures is essential to stability against a potential slippage failure.

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Table 1

Visual Classification of Soil Borings

Taken on Existing Test Levee

Elevation,* ft	Classification
107.0-106.0	Light tan silt
106.0-104.0	Gray silty clay
104.0-101.0	Silt with small clay pockets
101.0-98.0	Dark gray silt with sand
98.0-96.0	Dark gray silt

* Refer to Figure 1.

Table 2

Summary of Test Results for Expedient Levee-Raising Structures

Test Sections	Reference Plate No.	Structure Performance: Static Head	Structure Performance: Wave Action	Comments
Section 2-1; 2-ft-high potato ridge (clayey silt)	1	Very stable	Failed	Stable for static heads up to 1.5 ft; failed for 0.4-ft waves.
Section 2-2; 2-ft-high potato ridge with polyethylene covering	4	Very stable	Failed	Stable for static heads up to 1.5 ft; failed for 0.4-ft waves.
Section 2-3; 2-ft-high potato ridge (clay gravel)	7	Very stable	Failed	Stable for static heads up to 1.5 ft; failed for 0.4-ft waves.
Section 2-4; 2-ft-high potato ridge encased in 6 mil polyethylene	12	Very stable	Failed	Stable for static heads up to 1.5 ft; failed for 0.4-ft waves.
Section 2-5; 2-ft-high earth-filled sacks	15	Very stable	Stable	Stable for static heads up to 1.5 ft; 0.4-ft wave induced minor damage by pumping fill material through bags.
Section 4-1; 4-ft-high earth-filled sacks	17	Very stable	Stable	Stable for static heads up to 3.4 ft; 0.75-ft wave induced minor damage by pumping fill material out of bags.
Section 4-2; 4-ft-high plastic grid with sand fill (with filter)	19	Very stable	Stable	Stable for static heads up to 3.0 ft; 0.75-ft wave induced minor damage.
Section 4-2; 4-ft-high plastic grid with sand fill (without filter)	19	Stable	Failed	Stable for static head up to 3.0 ft, but sand fill leaches out of grid; failed under wave action due to leaching of sand fill.
Section 4-3; 3-ft-high plywood flashboard	20	Failed	--	Failed under 1.0-ft static head due to boils under plywood and around support posts.
Section 4-3-A; 3-ft-high plywood flashboard with 2-ft-high compacted earth fill	21	Very stable	Failed	Stable for static heads up to 2.9 ft; failed due to wave overtopping.
Section 4-3-B; 3-ft-high plywood flashboard with 1-ft-high compacted earth fill	22	Failed	--	Stable for static heads up to 2.0 ft; static heads above 2.0 ft caused boils and leaching under structure.
Section 4-3-C; 3-ft-high plywood flashboard with 2-ft-high uncompacted earth fill	23	Failed	--	Stable for static heads up to 2.5 ft; greater static heads cause boils and leaching.
Section 4-3-D; 3-ft-high plywood flashboard with asphalt sealer	24	Failed	--	Stable for static heads up to 2.0 ft; greater static heads caused boils and leaching.
Section 4-4; 3-ft-high planking flashboard with 2.8-ft-high uncompacted earth backing	25, 26	Very stable	Failed	Stable for static heads up to 2.5 ft; failed due to wave overtopping.

(Continued)

Table 2 (Concluded)

Test Sections	Reference Plate No.	Structure Performance Static Head	Structure Performance Wave Action	Comments
Section 4-5; 4-ft-high plywood mud box filled with earth	27	Failed	--	Failed under 1.0 ft static head due to boils and leaching under structure.
Section 4-6; 4-ft-high planking mud box with tamped earth fill	28	Very stable	Stable	Stable for static heads up to 3.5 ft; stable for 0.75-ft waves as long as wave overtopping does not occur; though damage rate slow, failure inevitable if wave overtopping occurs.
Section 4-6-A; modified 4-ft-high planking mud box with tamped earth fill	28	Stable	Adequate	Stable for static heads up to 3.5 ft; design is adequate for mild wave climate, but damage rate is faster than Section 4-6 when exposed to wave overtopping.
Section 6-1; 6-ft-high plastic grid with sand fill and spun-woven filter fabric between lifts	29	Stable	Failed	Stable for static heads up to 4.0 ft; static heads above this cause structural problems; stable for 0.75-ft waves as long as swl does not exceed 3 ft.
Section 6-2; 5-ft-high planking mud box with tamped earth fill	30	Very stable	Failed	Stable for static heads up to 4.5 ft; stable design for 0.75-ft waves as long as no wave overtopping occurs; wave overtopping will result in structural failure.
Section 6-3; 6-ft-high, 2-in. entrenched plywood mud box with earth fill	31	Stable	--	Stable for static heads not exceeding 1.0 ft; static heads exceeding this will result in undermining and failure of structure.
Section 6-3-A; 6-ft-high, 2-in. entrenched plywood mud box with earth fill and 4-in. riverside earth berm	32	Stable	--	Stable for static heads not exceeding 4.0 ft; structure's earth fill failed when exposed to 5.0-ft static head; not recommended design; should be constructed in field as shown on Plate 33, Section 6-4.
Section 6-4; 6-ft-high, 6-in. entrenched plywood mud box with earth fill	33	Stable	--	Stable with minor seepage for static heads not exceeding 2.0 ft; earth fill failed after 4.0 hrs of 3.0-ft static head.
Section 6-5; modified 6-ft-high, 6-in. entrenched plywood mud box with earth fill	34	Stable	Stable	Very stable for static heads not exceeding 3.0 ft; some minor damage and seepage for static heads up to 5.0 ft; stable design for 0.75-ft waves as long as wave overtopping minimal.

Photo 1. Small test basin and equipment

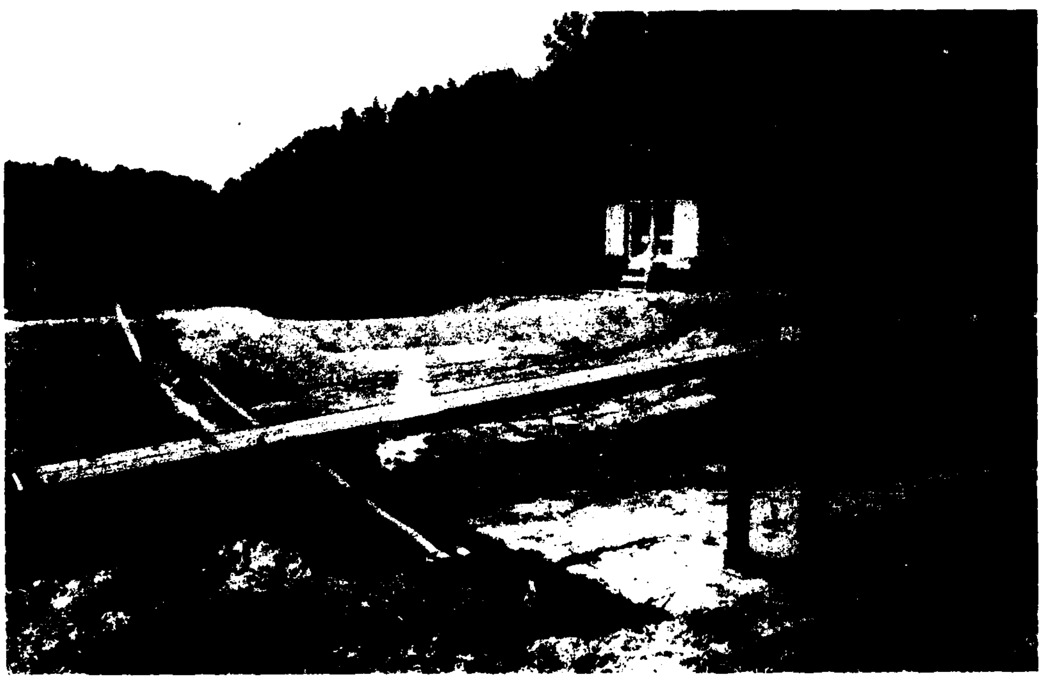

Photo 2. Small and large test basins and equipment

Photo 3. Wave generator and moveable framework

Photo 4. Nuclear probe for measuring in-place moisture content and soil densities

Photo 5. Riverside view of Section 2-1 before testing

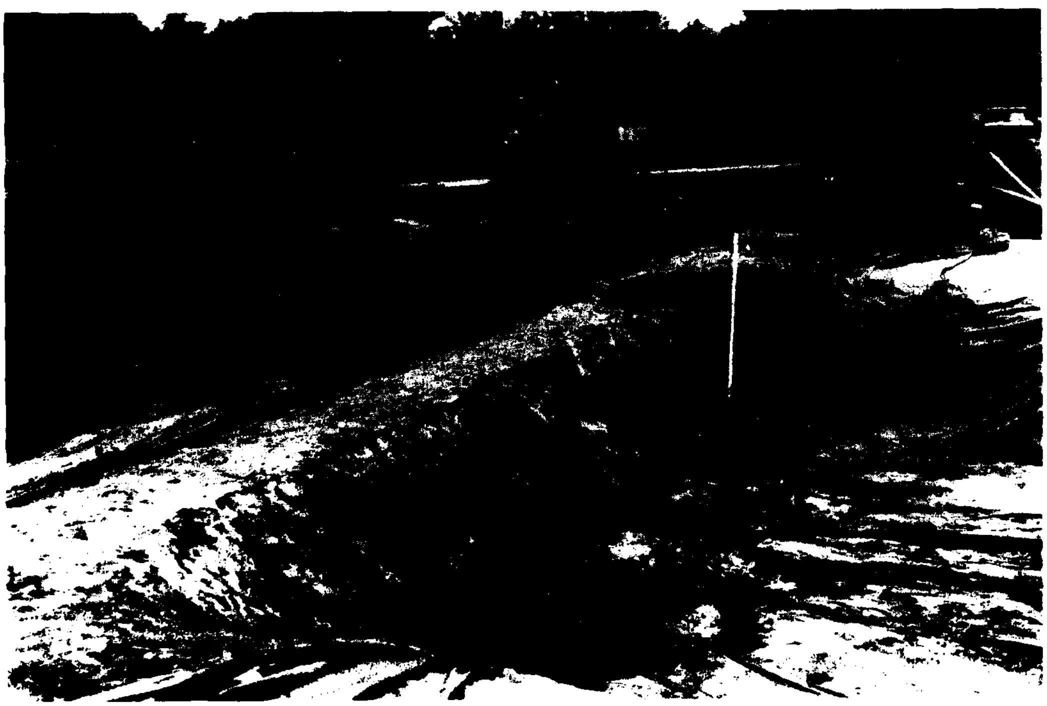

Photo 6. Landside view of Section 2-1 before testing

Photo 7. Soil compaction during construction of Section 2-1

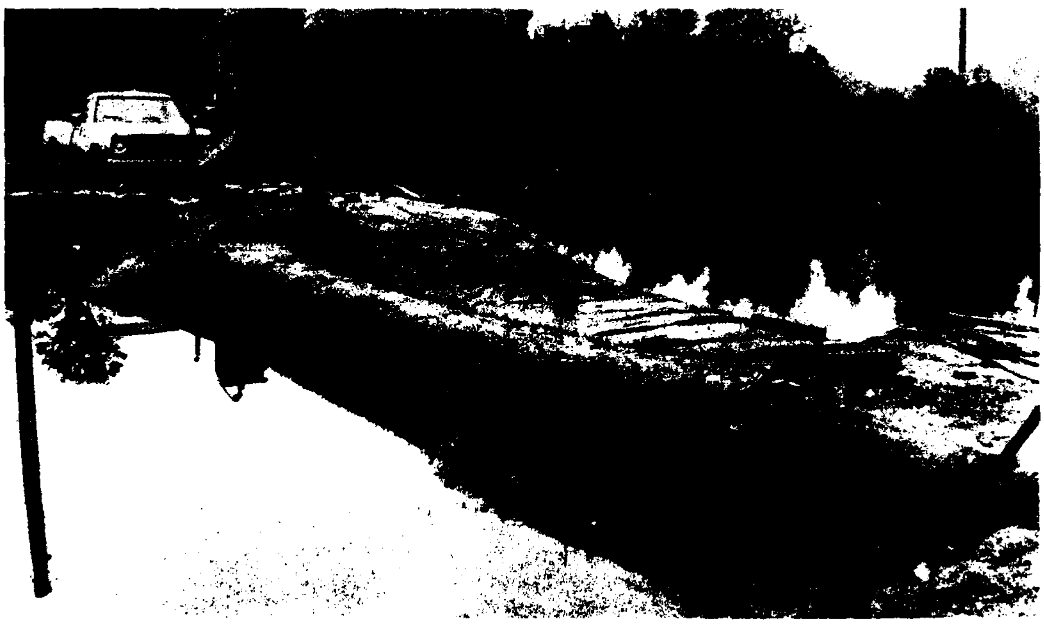

Photo 8. Riverside view of Section 2-1 after testing 0.5-ft static differential head

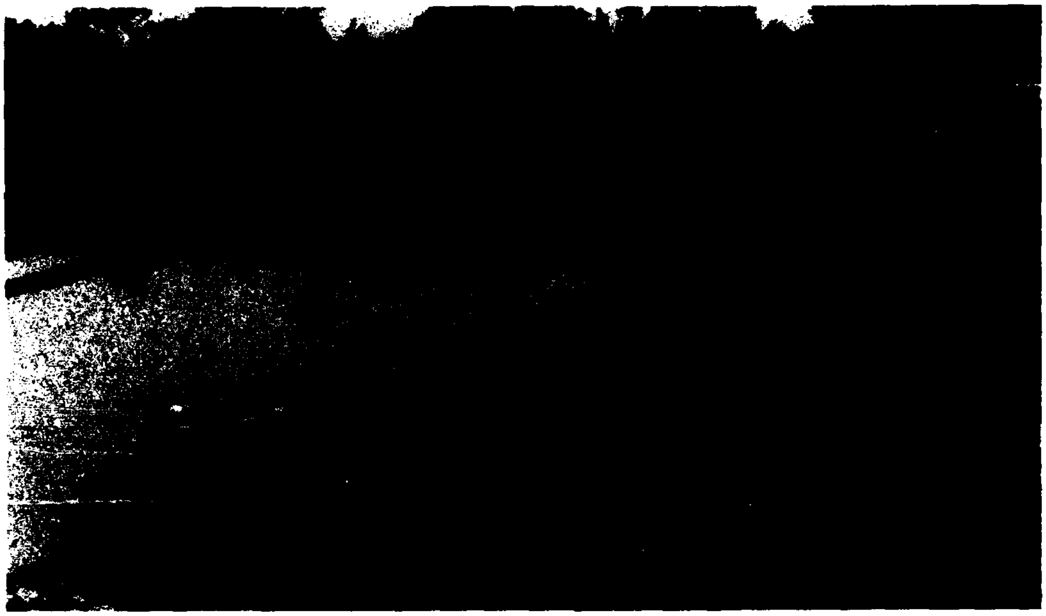

Photo 9. Landside view of Section 2-1 after testing 0.5-ft static differential head

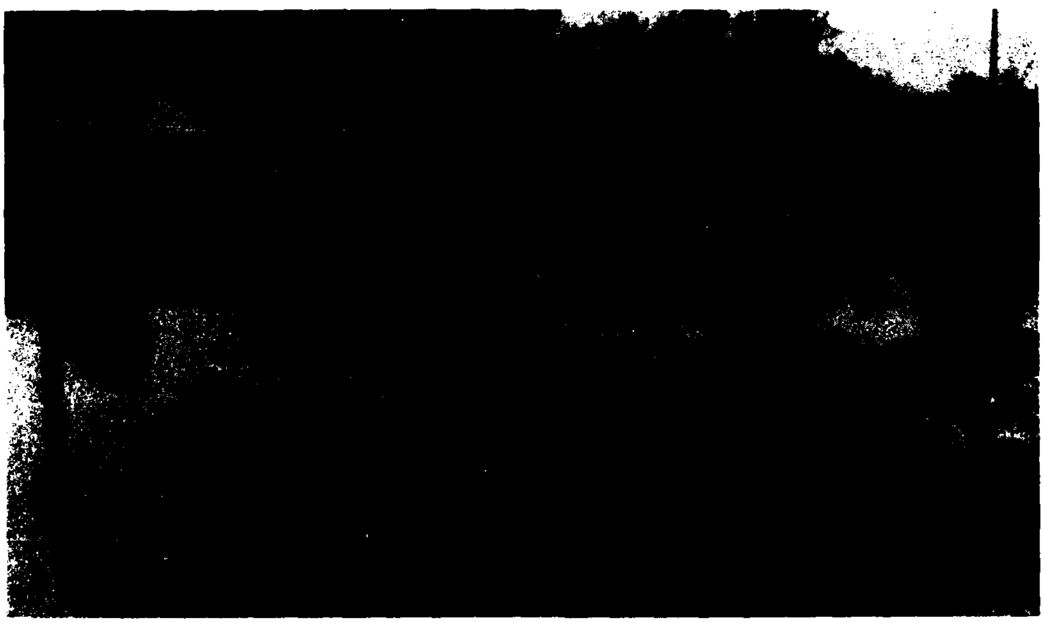

Photo 10. Riverside view of Section 2-1 after testing 1.0-ft static differential head

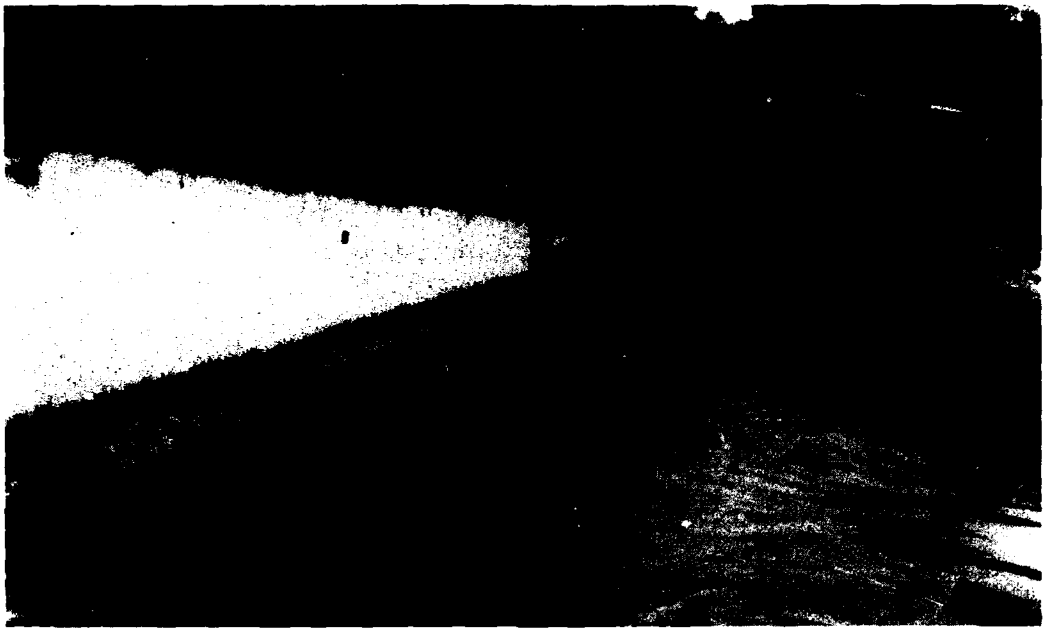

Photo 11. Landside view of Section 2-1 after testing 1.0-ft static differential head

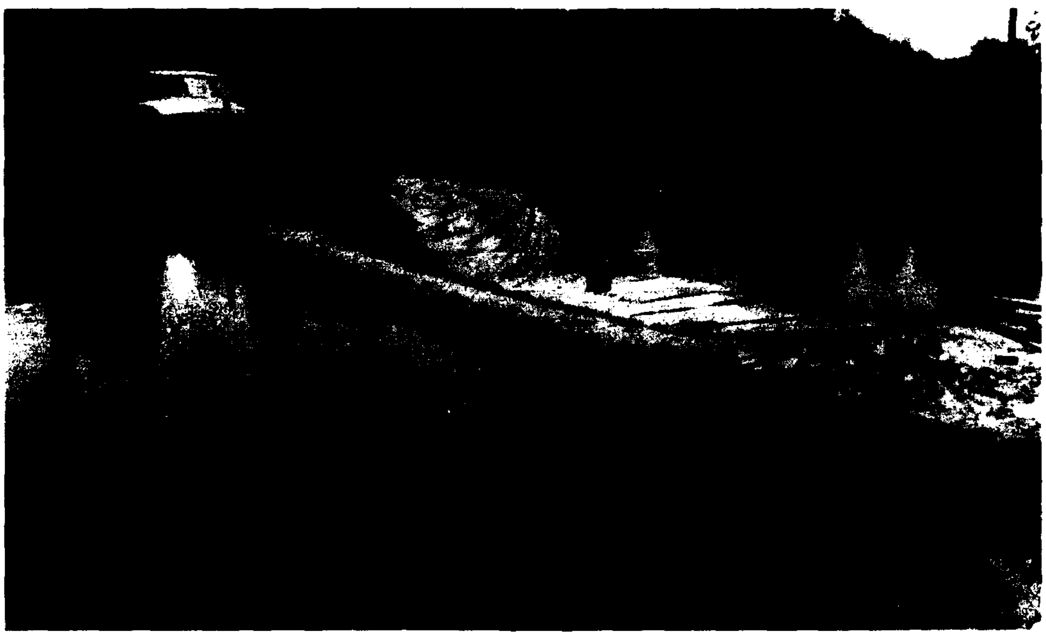

Photo 12. Riverside view of Section 2-1 after testing 1.5-ft static differential head

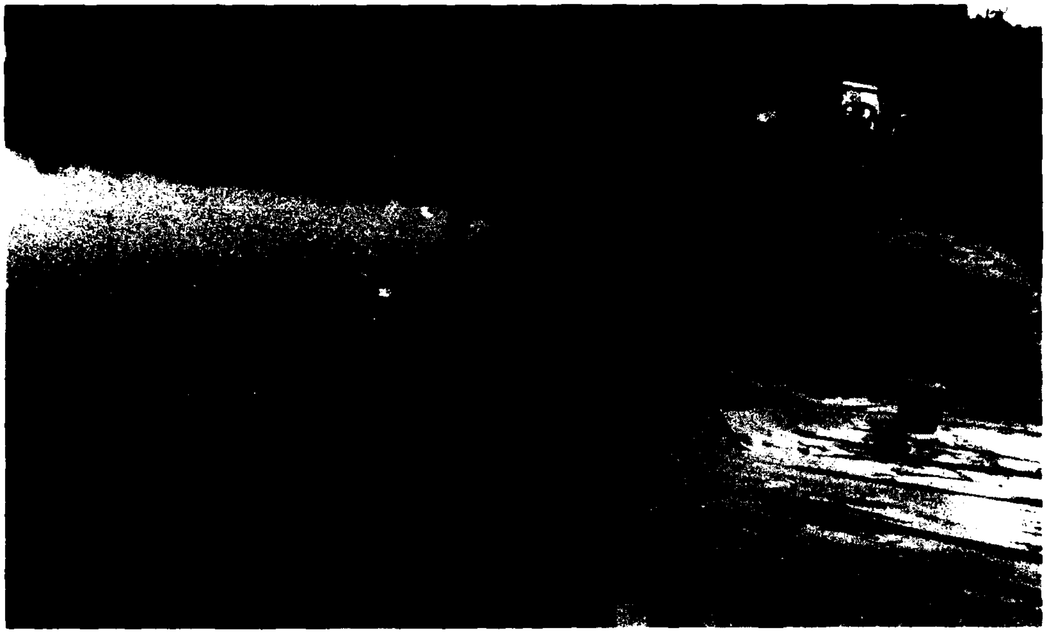

Photo 13. Landside view of Section 2-1 after testing 1.5-ft static differential head

Photo 14. Riverside view of Section 2-1 after testing 2.5 hr of wave action

Photo 15. Riverside view of Section 2-1 after testing 8.0 hr of wave action

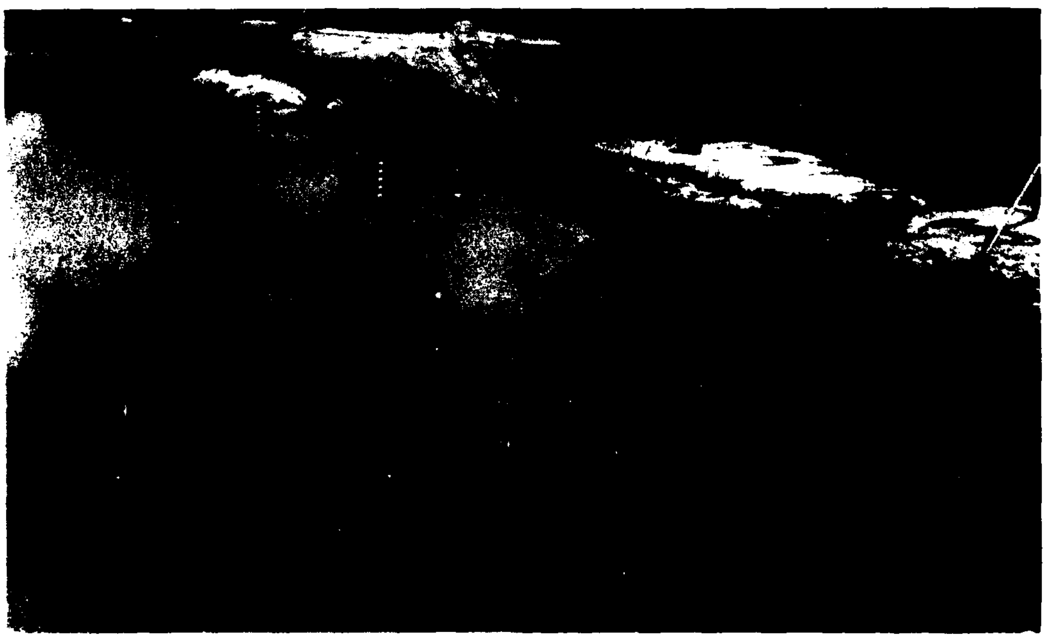

Photo 16. Riverside view of Section 2-1 after testing 10.75 hr of wave action

Photo 17. Riverside view of Section 2-1 after testing 19 hr of wave action (end of test)

Photo 18. End view of Section 2-1 after testing 19 hr of wave action (end of test)

Photo 19. Landside view of Section 2-1 after testing 19 hr of wave action (end of test)

Photo 20. Riverside view of Section 2-1 at end of test (small basin drained)

Photo 21. Riverside view of Section 2-2 before testing

Photo 22. Landside view o' Section 2-2 before testing

Photo 23. Side view of Section 2-2 after 0.5 hr of wave action at the 1.0-ft water level

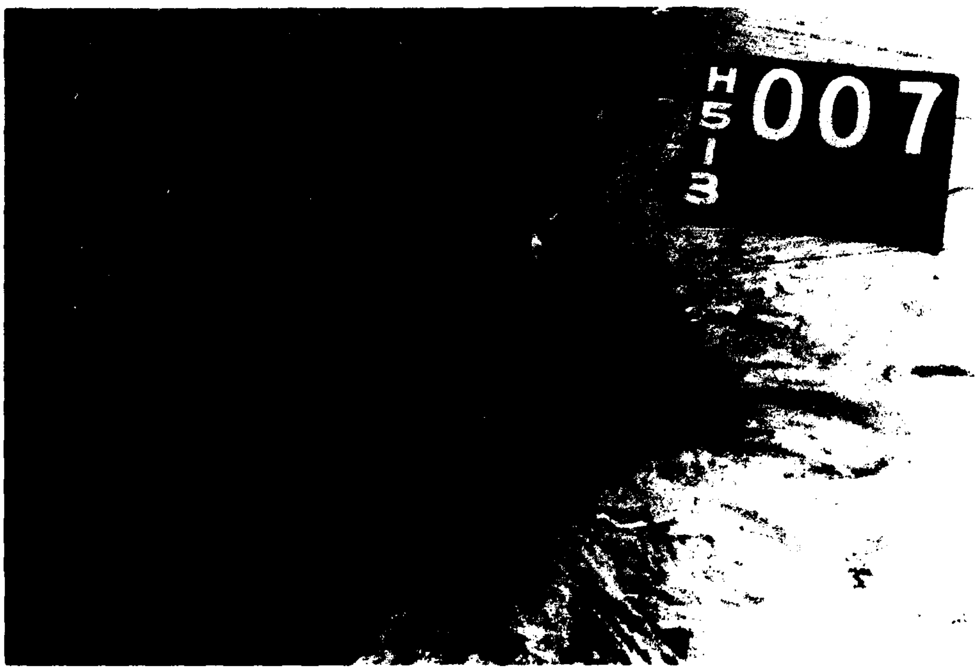

Photo 24. Close-up of sand bags placed over the riverside of the lap joint after 2.0 hr of wave action at the 1.0-ft water level

Photo 25. Side view of Section 2-2 after 10.0 hr of wave action at the 1.0-ft water level

Photo 26. Riverside view of Section 2-2 after 19 hr of wave action at the 1.0-ft water level

Photo 27. Side view of Section 2-2 after 19 hr of wave action at the 1.0-ft water level

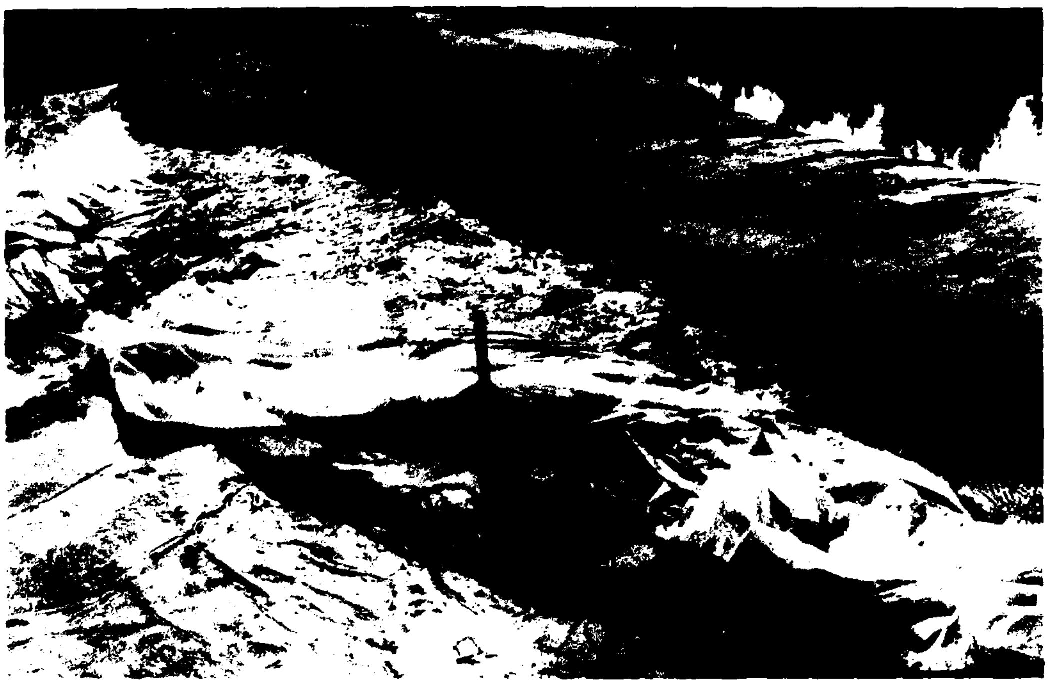

Photo 28. Riverside view of Section 2-2 after 19 hr of wave action at the 1.0-ft water level (poly covering removed)

Photo 29. Side view of Section 2-2 after 19 hr of wave action at the 1.0-ft water level (poly covering removed)

Photo 30. Riverside view of Section 2-2 after 34.5 hr of wave action at the 1.0-ft water level

Photo 31. Landside view of Section 2-2 after 34.5 hr of wave action at the 1.0-ft water level

Photo 32. Landside view of Section 2-2 after 34.5 hr of wave action at the 1.0-ft water level (poly covering removed)

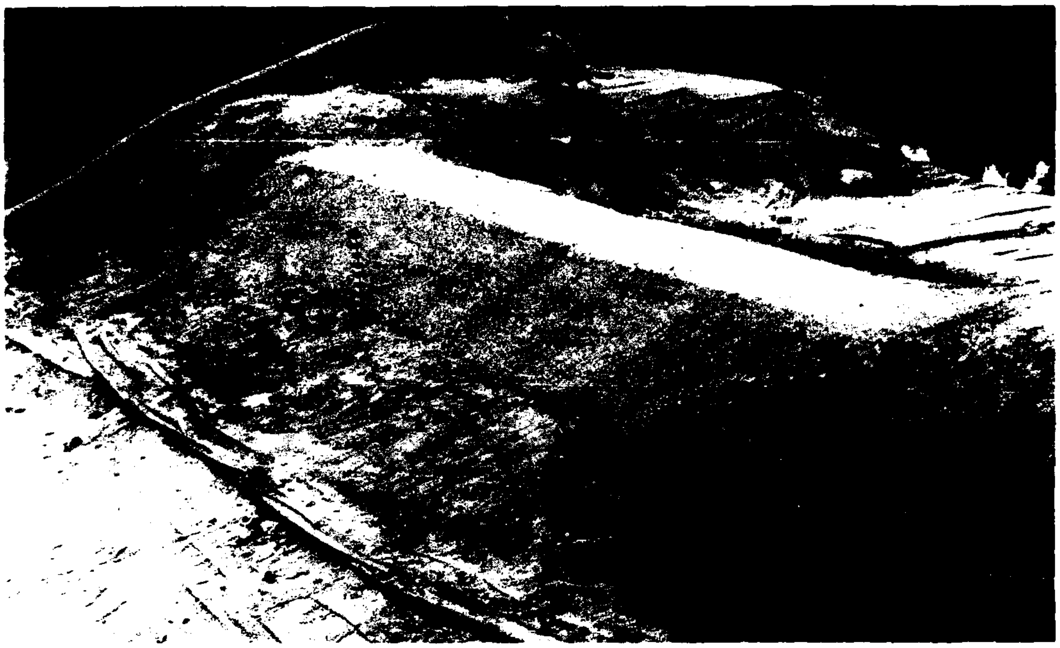

Photo 33. Riverside view of Section 2-3 before testing

Photo 34. Landside view of Section 2-3 before testing

Photo 35. Riverside view of Section 2-3 after testing 0.5-, 1.0-, and 1.5-ft static differential heads

Photo 36. Landside view of Section 2-3 after testing 0.5-, 1.0-, and 1.5-ft static differential heads

Photo 37. Riverside view of Section 2-3 after 2.5 hr of wave action at the 1.0-ft water level

Photo 38. Riverside view of Section 2-3 after 10.5 hr of wave action at the 1.0-ft water level

Photo 39. Riverside view of Section 2-3 after 19 hr of wave action at the 1.0-ft water level

Photo 40. Landside view of Section 2-3 after 19 hr of wave action at the 1.0-ft water level

Photo 41. Riverside view of Section 2-3 after 28 hr of wave action at the 1.0-ft water level

Photo 42. Riverside view of Section 2-3 after 12.5 hr of wave action at the 1.3-ft water level

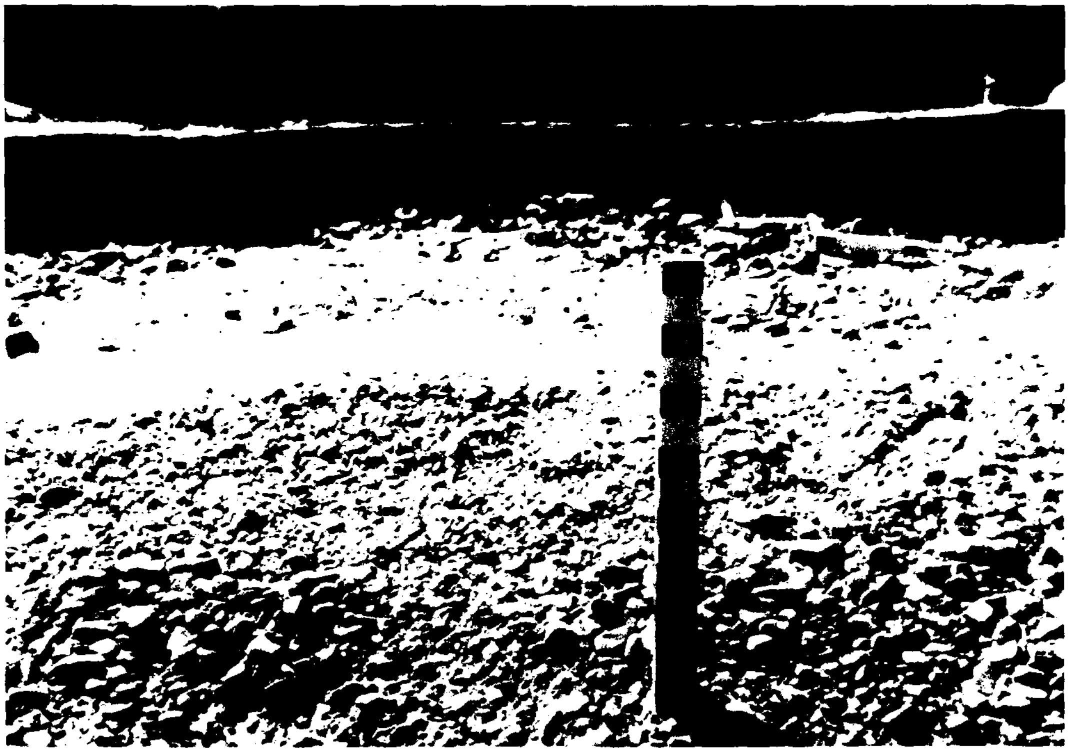

Photo 43. Close-up of gravel and sand beach formed on riverside of Section 2-3

Photo 44. Riverside view of Section 2-3 after 37 min of wave action at the 1.4 ft water level (end of test)

Photo 45. Landside view of Section 2-3 after 37 min of wave action at the 1.4-ft water level (end of test)

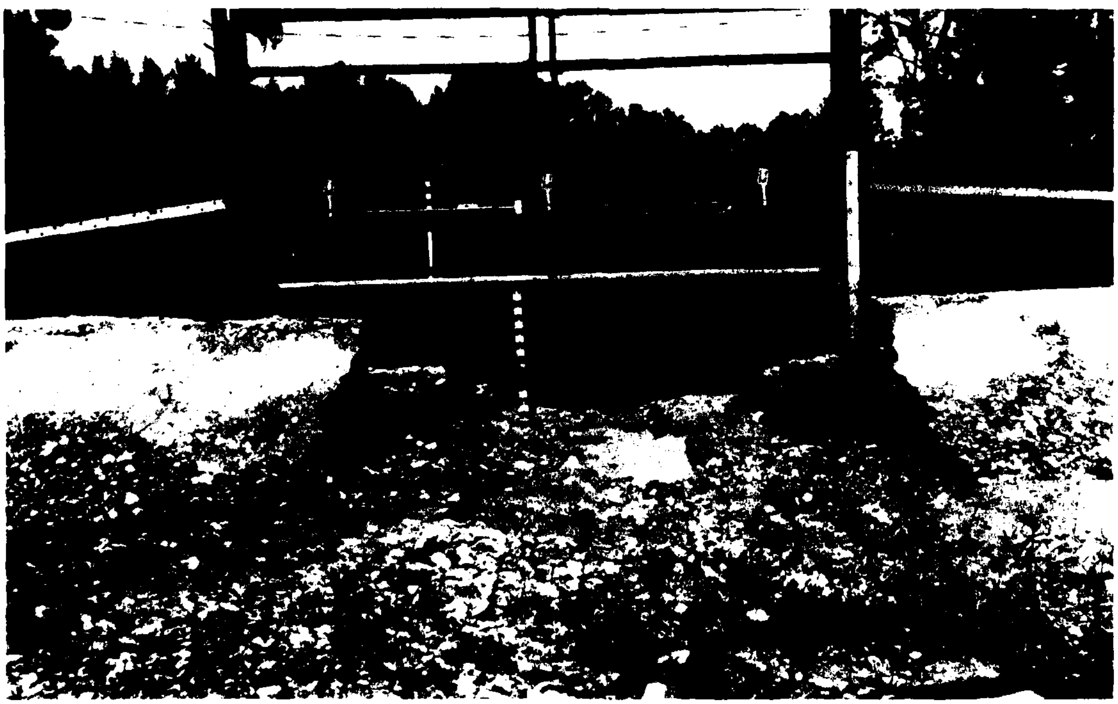

Photo 46. Close-up of breech through Section 2-3 (end of test)

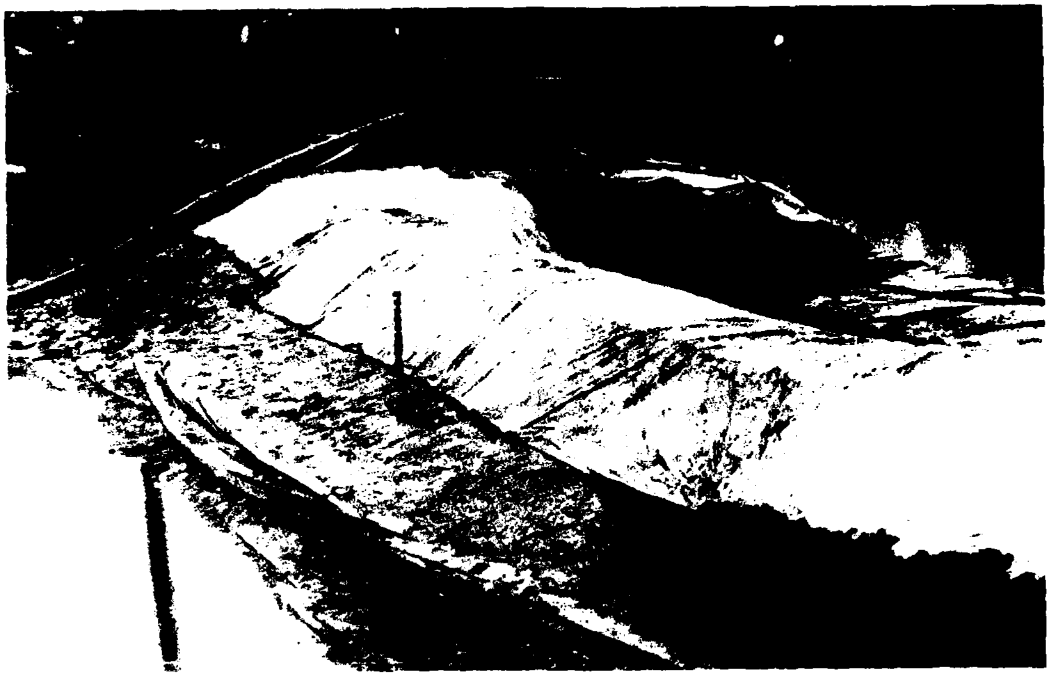

Photo 47. Riverside view of Section 2-4 before testing

Photo 48. Landside view of Section 2-4 before testing

Photo 49. Riverside view of Section 2-4 after testing 0.5-, 1.0-, and 1.5-ft static differential heads

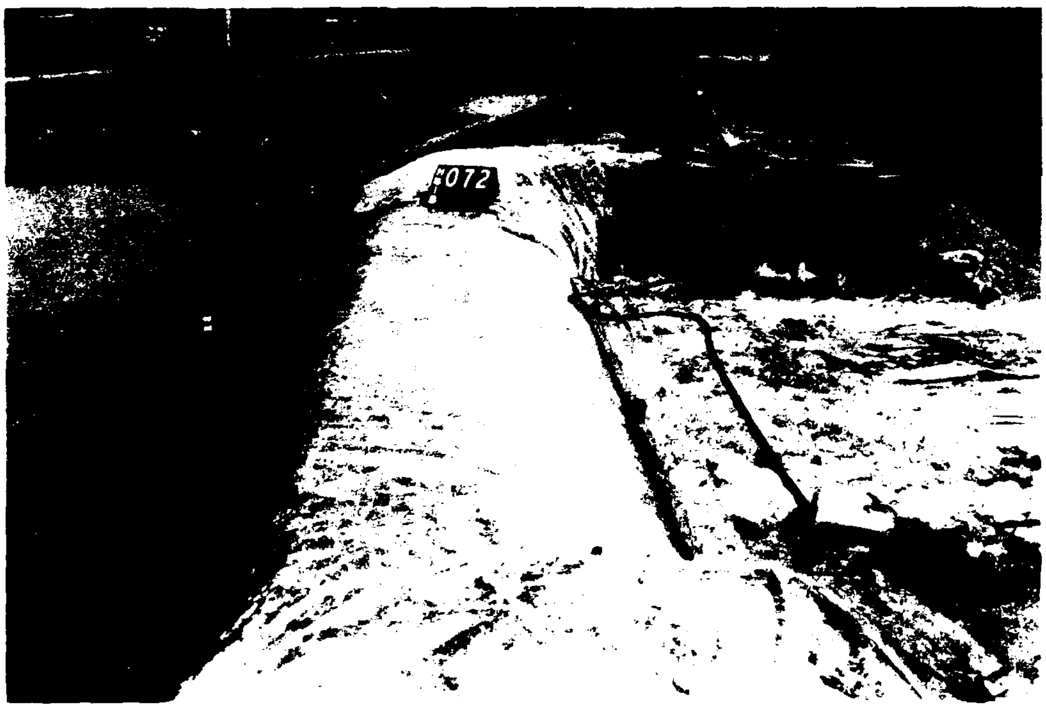

Photo 50. Side view of Section 2-4 after testing 0.5-, 1.0-, and 1.5-ft static differential heads

Photo 51. Riverside view of Section 2-4 after 19 hr of wave action at the 1.0-ft water level

Photo 52. Side view of Section 2-4 after 19 hr of wave action at the 1.0-ft water level

Photo 53. Landside view of Section 2-4 after 14.5 hr of wave action at the 1.3-ft water level

Photo 54. Side view of Section 2-4 after 18 hr of wave action at the 1.3-ft water level

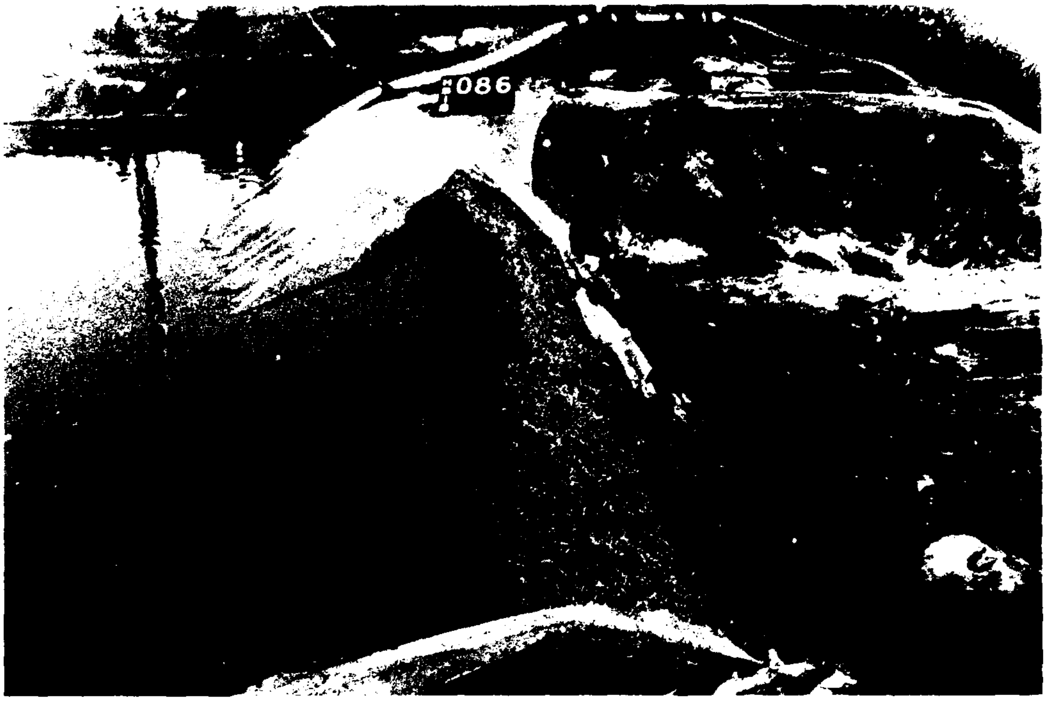

Photo 55. Side view of Section 2-4 after 25.5 hr of wave action at the 1.3-ft water level

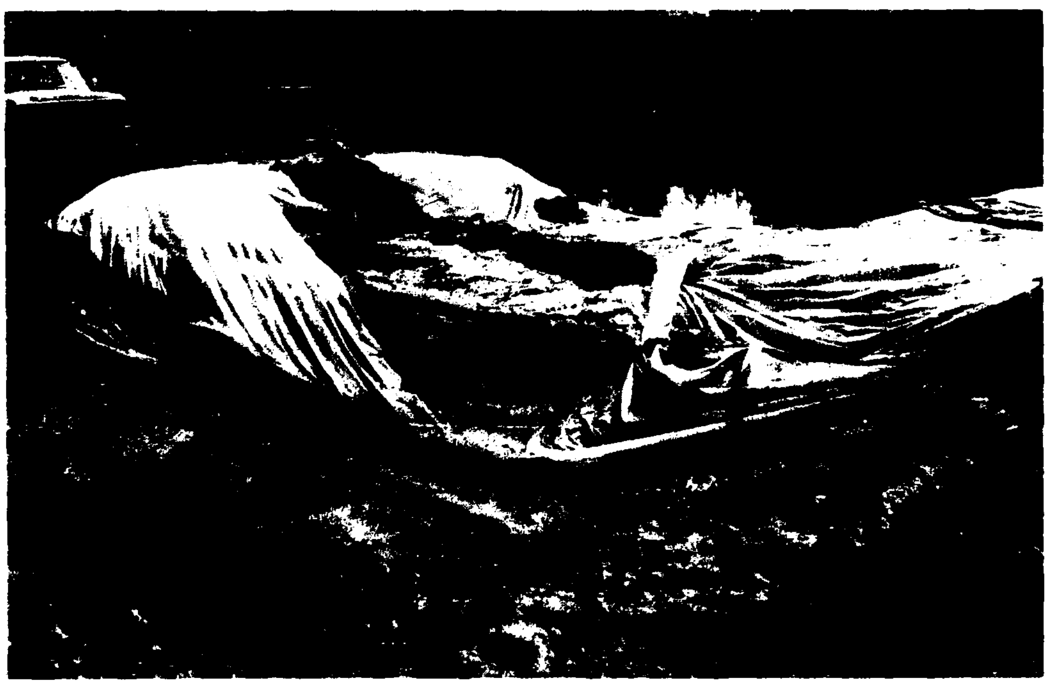

Photo 56. Riverside view of Section 2-4 at end of test

Photo 57. Side view of Section 2-4 at end of test

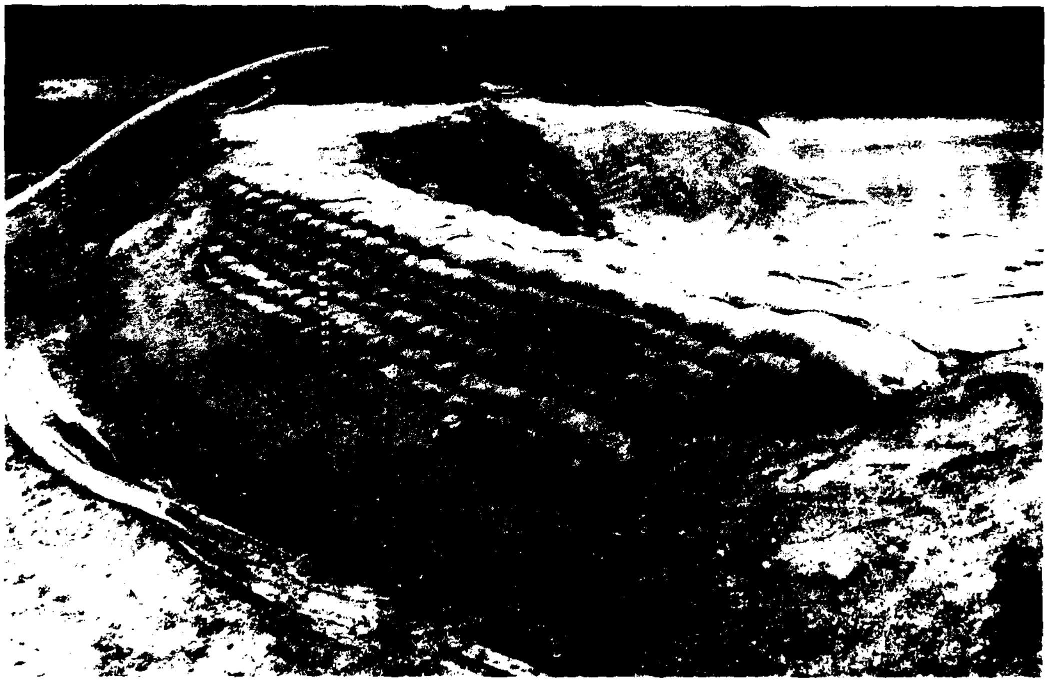

Photo 58. Riverside view of Section 2-5 before testing

Photo 59. Landside view of Section 2-5 before testing

Photo 60. Wave action on Section 2-5 (water level = 1.0 ft)

Photo 61. Riverside view of Section 2-5 after 19 hr of wave action at the 1.0-ft water level

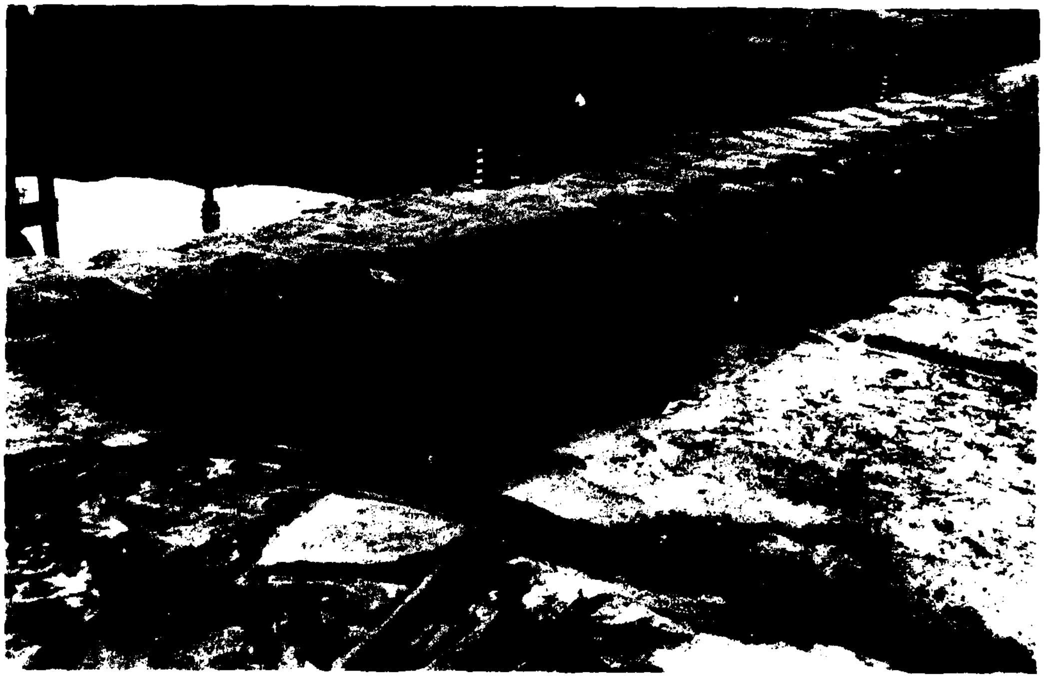

Photo 62. Landside view of Section 2-5 after 19 hr of wave action at the 1.0-ft water level

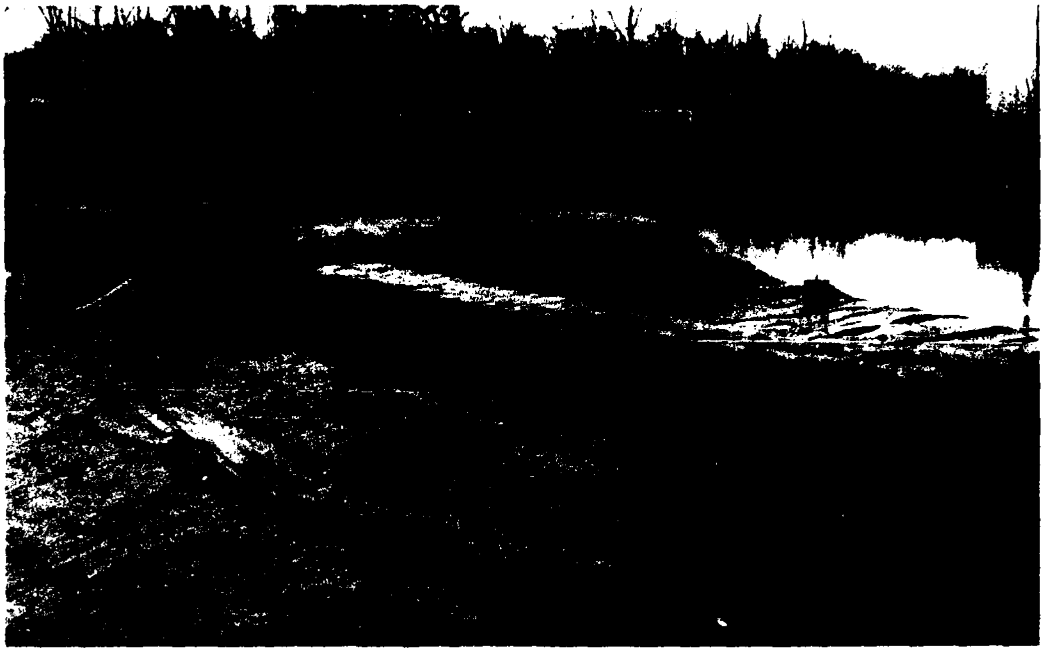

Photo 63. Riverside view of Section 2-5 after 19 hr of wave action at the 1.3-ft water level (end of test)

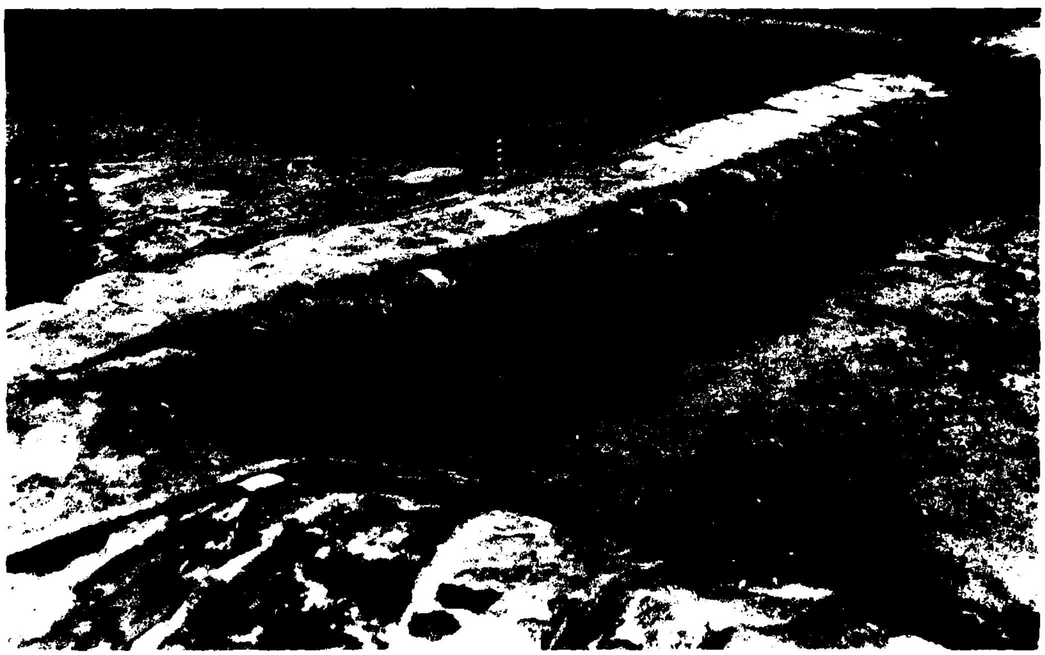

Photo 64. Landside view of Section 2-5 after 19 hr of wave action at the 1.3-ft water level (end of test)

Photo 65. Riverside view of Section 4-1 before testing

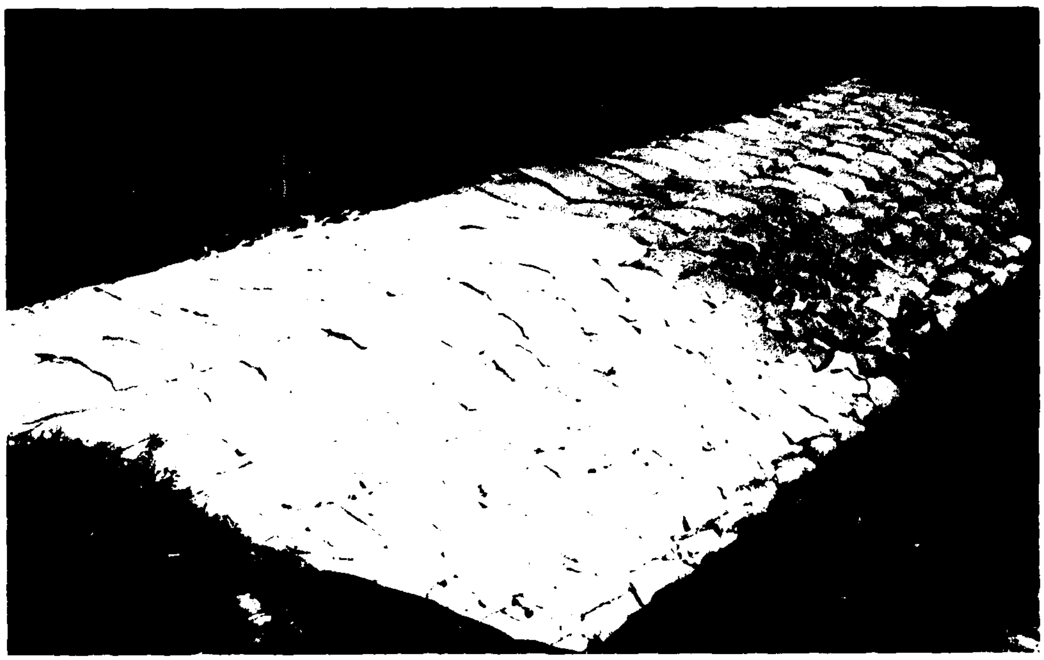

Photo 66. Landside view of Section 4-1 before testing

Photo 67. Riverside view of Section 4-1 after testing 1.1- to 2.0-ft static differential head

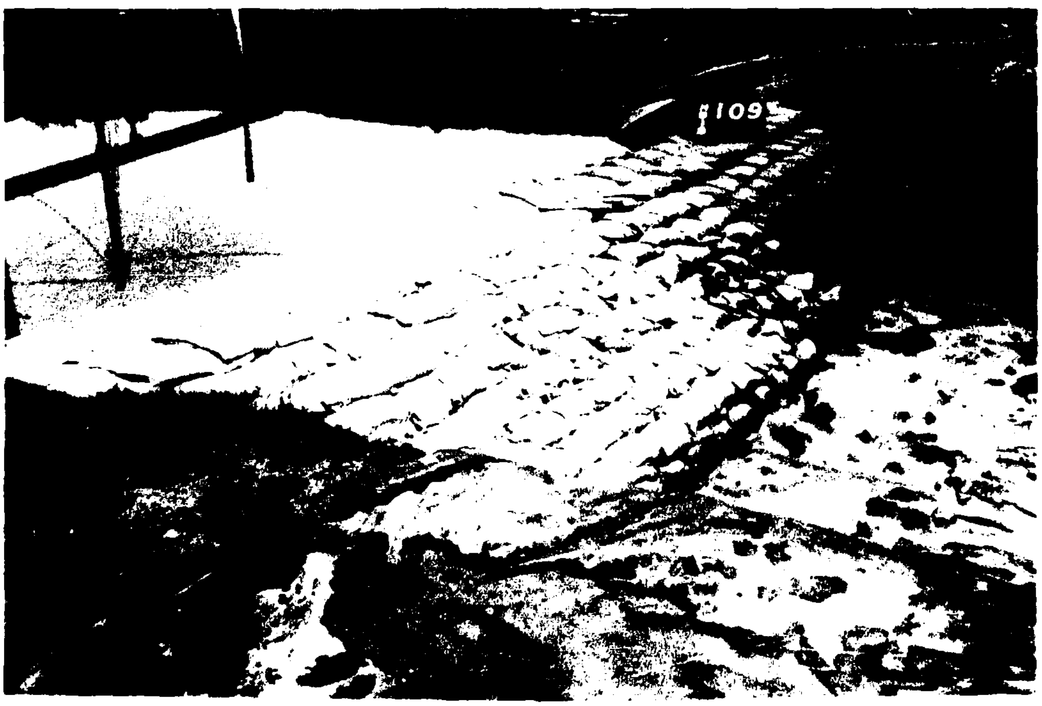

Photo 68. Landside view of Section 4-1 after testing 1.1- to 2.0-ft static differential head

Photo 69. Seepage occurring through the woven and spun woven polypropylene sacks at the end of the 3.0- to 3.35-ft static differential head

Photo 70. Seepage occurring through the burlap sacks at the end of the 3.0- to 3.35-ft static differential head

Photo 71. Riverside view of Section 4-1 at the end of the static differential head tests

Photo 72. Landside view of Section 4-1 at the end of the static differential head tests

Photo 73. Riverside view of Section 4-1 after 19 hr of wave action at the 1.0-ft water level

Photo 74. Riverside view of burlap sacks on Section 4-1 after 19 hr of wave action at the 1.0-ft water level

Photo 75. Riverside view of spun woven polypropylene sacks on Section 4-1 after 19 hr of wave action at the 1.0-ft water level

Photo 76. Riverside view of woven polypropylene sacks on Section 4-1 after 19 hr of wave action at the 1.0-ft water level

AD-A194 519 EFFECTIVENESS OF EXPEDIENT LEVEE-RAISING STRUCTURES: EXPERIMENTAL MODEL INVESTIGATION(U) COASTAL ENGINEERING RESEARCH CENTER VICKSBURG MS D G MARKLE ET AL. APR 88

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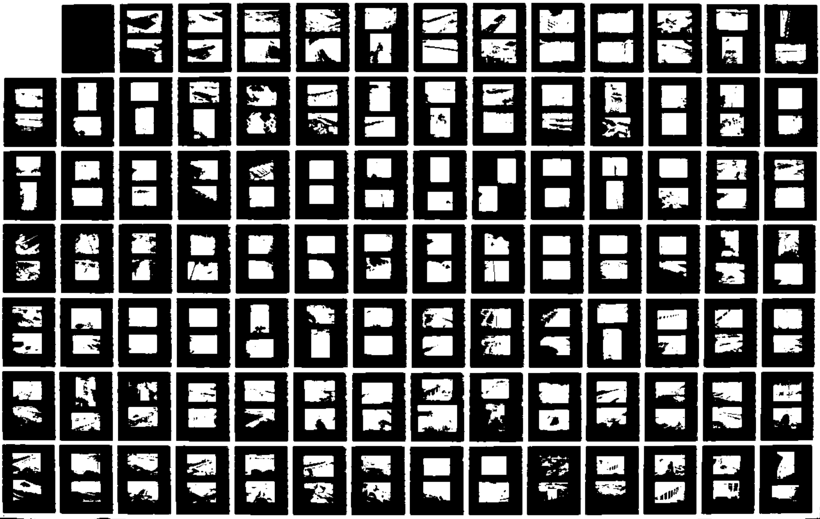

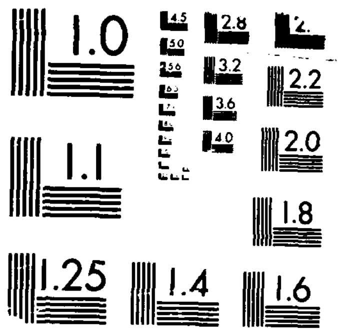

MICROCOPY RESOLUTION TEST CHART

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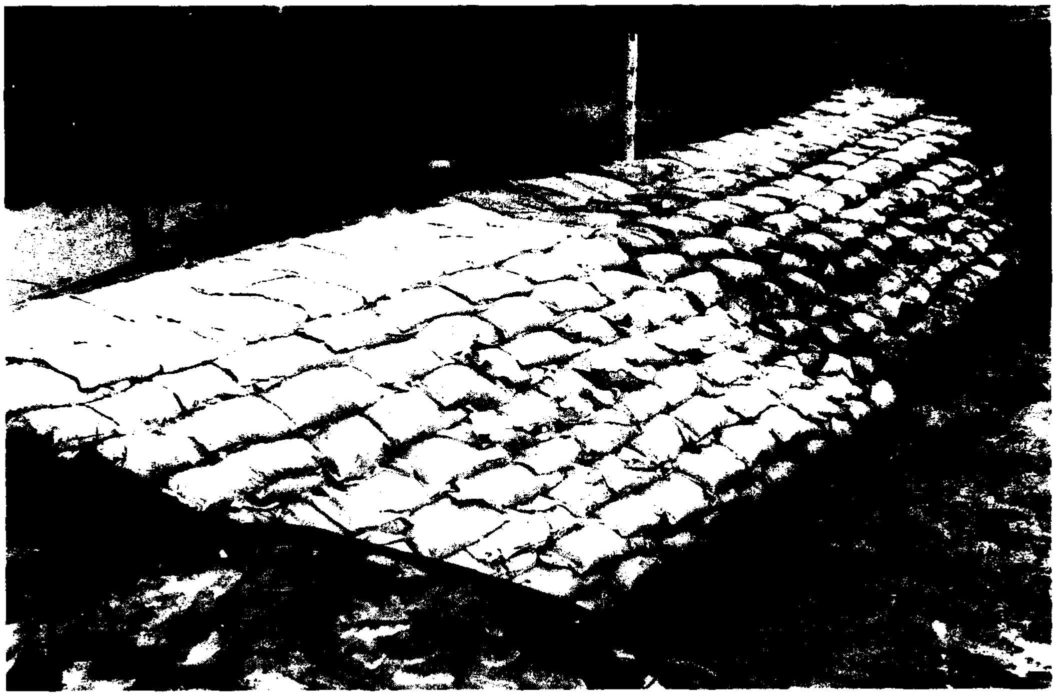

Photo 77. Landside view of Section 4-1 after 19 hr of wave action at the 1.0-ft water level

Photo 78. Riverside view of Section 4-1 during wave attack at the 2.0-ft water level

Photo 79. Riverside view of Section 4-1 after 19 hr of wave attack at the 2.0-ft water level

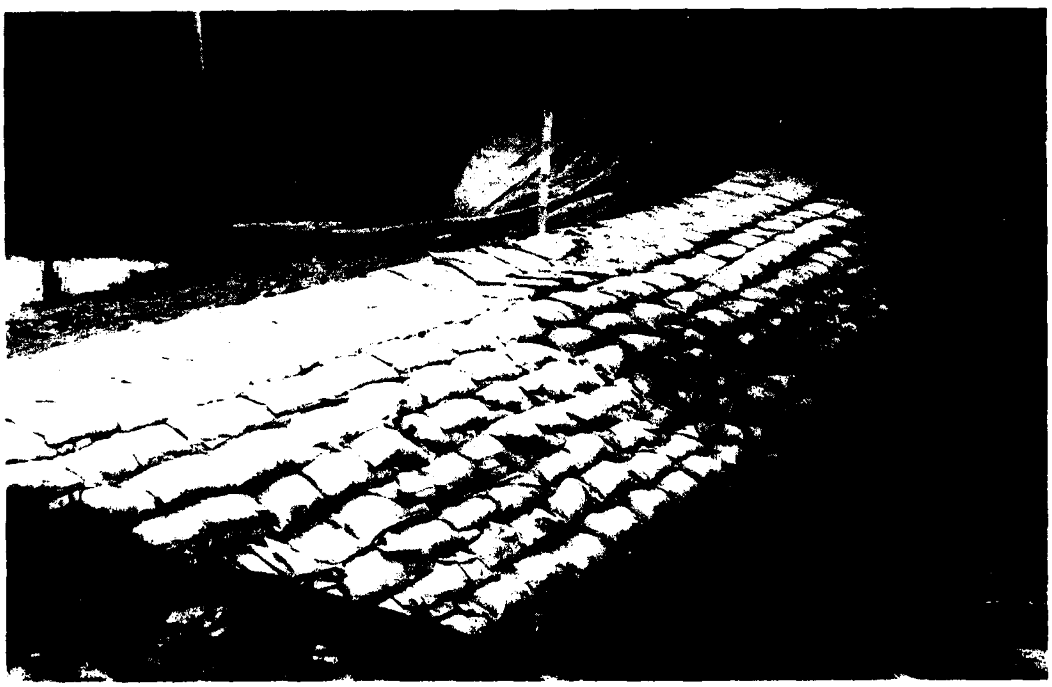

Photo 80. Landside view of Section 4-1 after 19 hr of wave attack at the 2.0-ft water level

Photo 81. Riverside view of spun woven polypropylene sacks on Section 4-1 after 19 hr of wave action at the 2.0-ft water level

Photo 82. Riverside view of Section 4-1 during wave attack at the 3.0-ft water level

Photo 83. Riverside view of Section 4-1 after 8 hr of wave action at the 3.0-ft water level (end of entire test)

Photo 84. Landside view of Section 4-1 during wave attack at the 3.0-ft water level

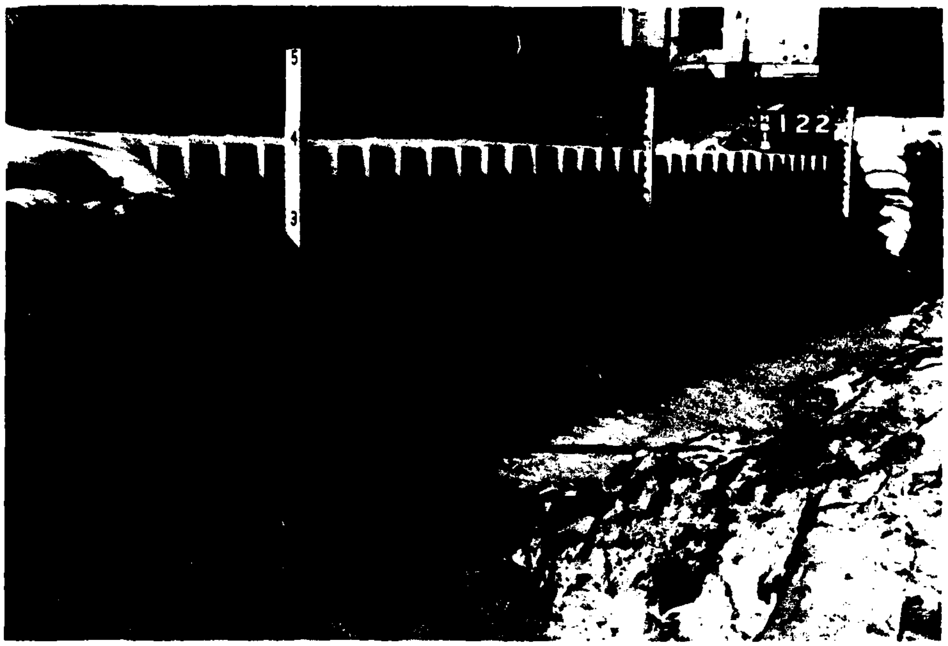

Photo 85. Riverside view of Section 4-2 before testing

Photo 86. End view of Section 4-2
before testing

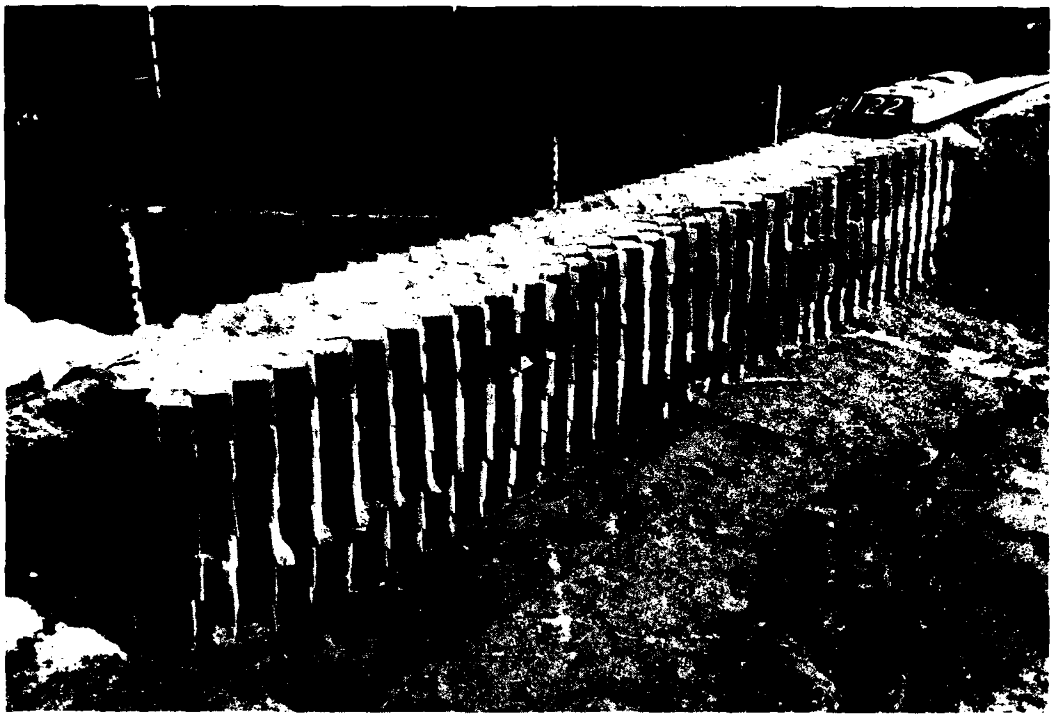

Photo 87. Landside view of Section 4-2 before testing

Photo 88. Unexpanded plastic grid

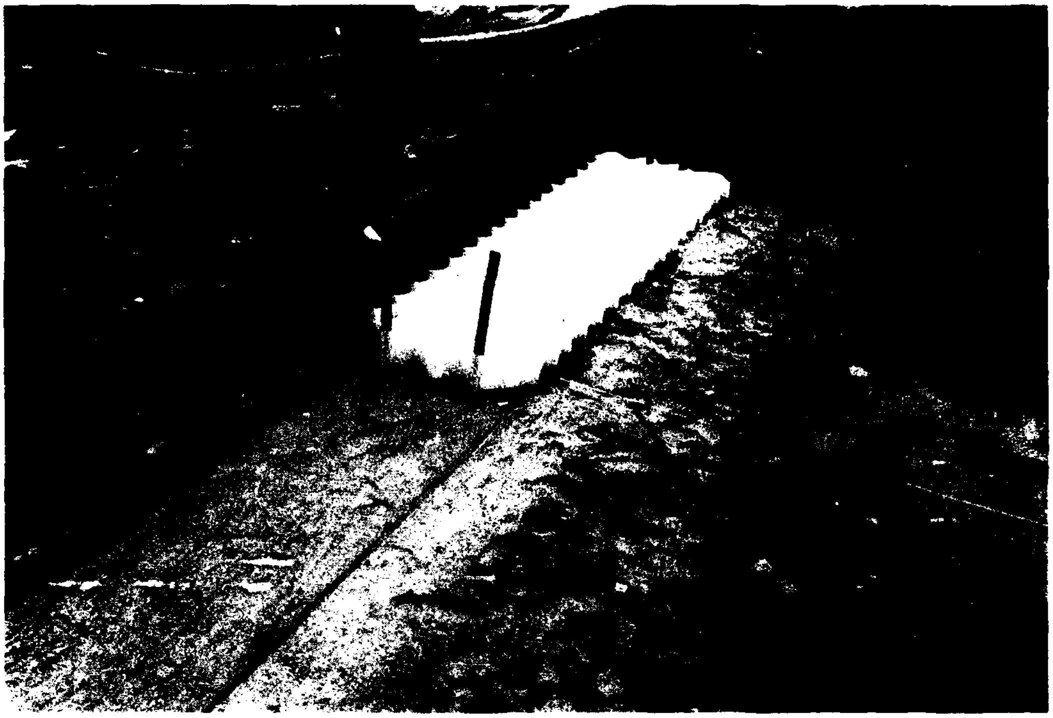

Photo 89. Expanded plastic grid in first lift of Section 4-2

Photo 90. Placement of sand fill in plastic grid

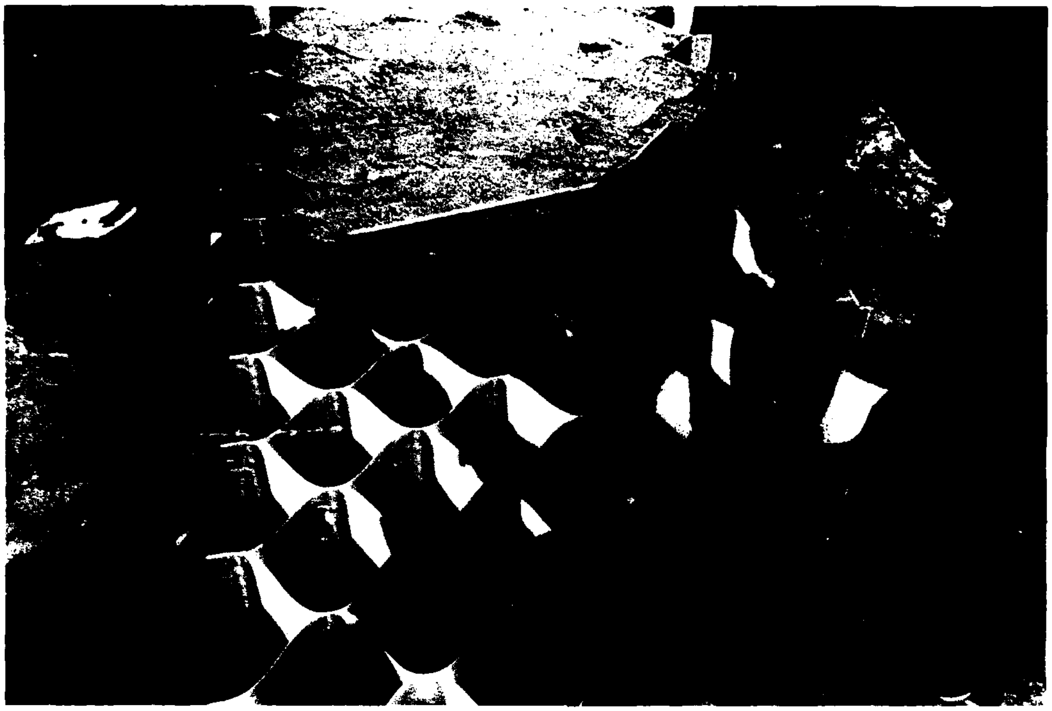

Photo 91. Screeding of excess sand fill

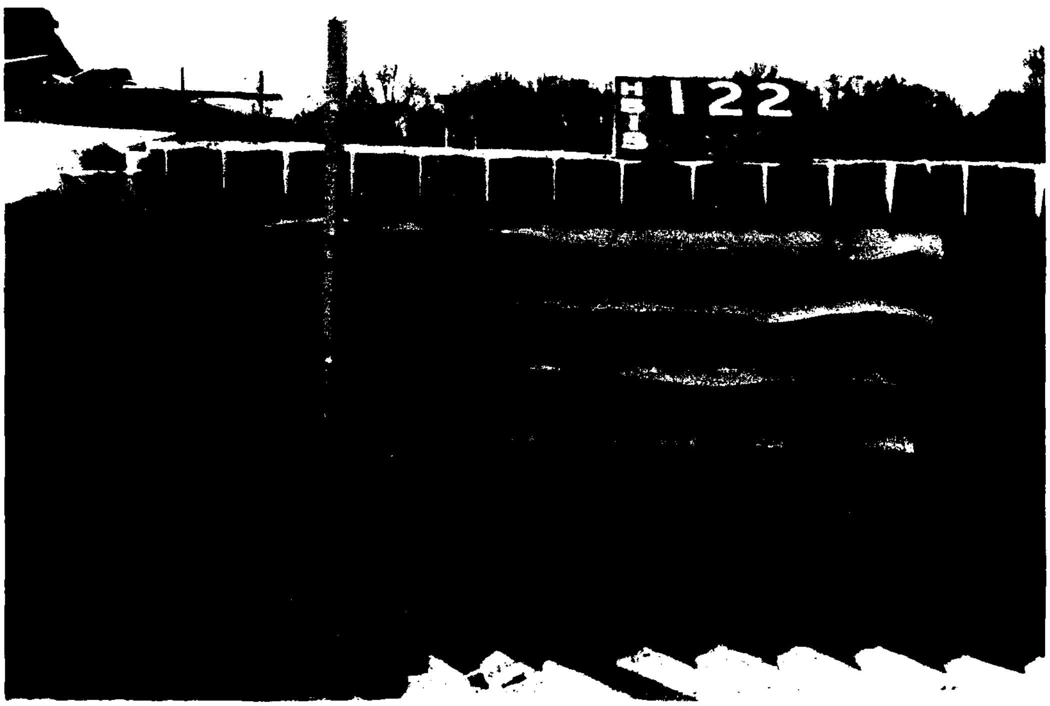

Photo 92. Riverside view of Section 4-2 with filter fabric between lifts

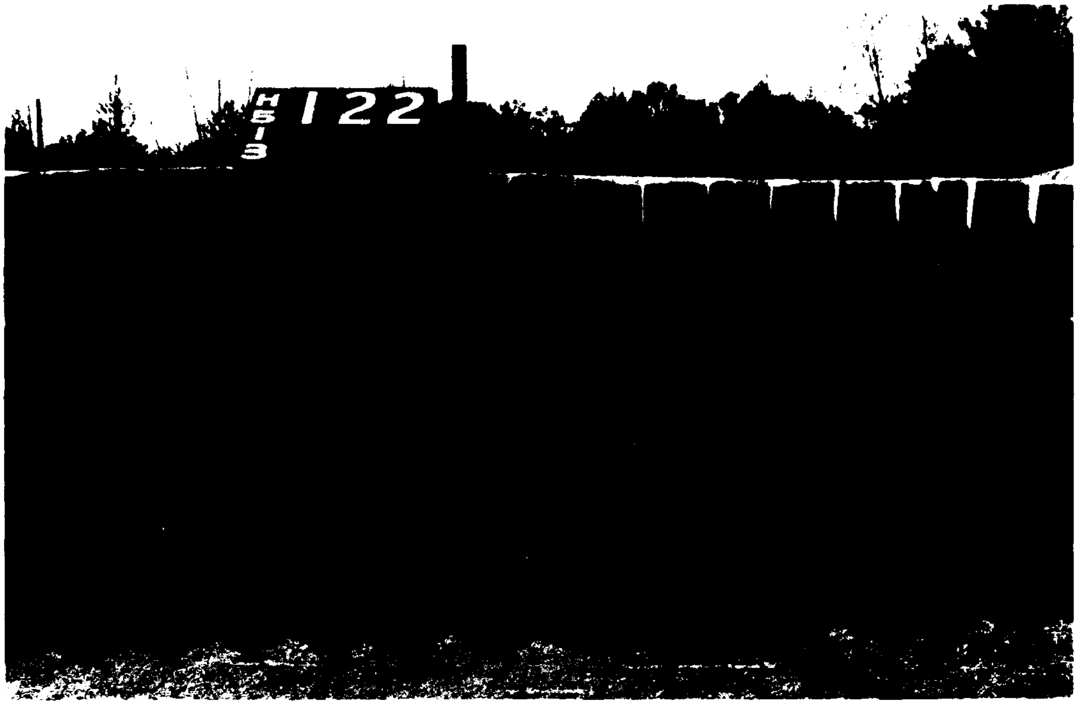

Photo 93. Riverside view of Section 4-2 with no filter between lifts

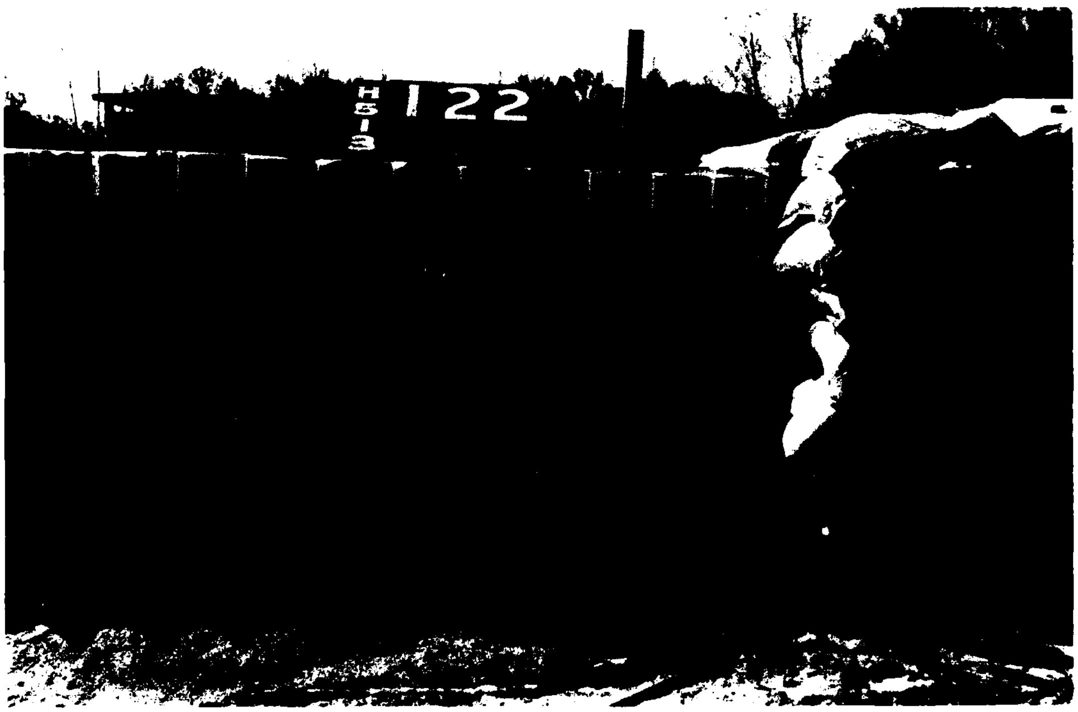

Photo 94. Riverside view of Section 4-2 with burlap sacks between lifts

Photo 95. View of Section 4-2 during construction showing the extent that the burlap and filter fabric extended into the structure

Photo 96. Riverside view of Section 4-2 at the end of the 2.0-ft static differential head

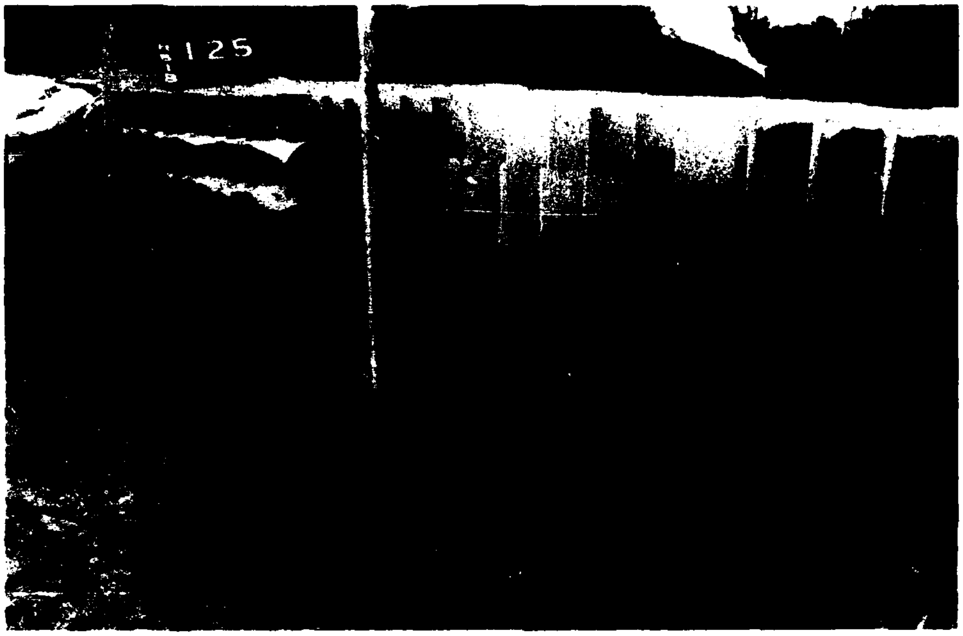

Photo 97. Riverside closeup view of the unfiltered portion of Section 4-2 at the end of the 2.0-ft static differential head

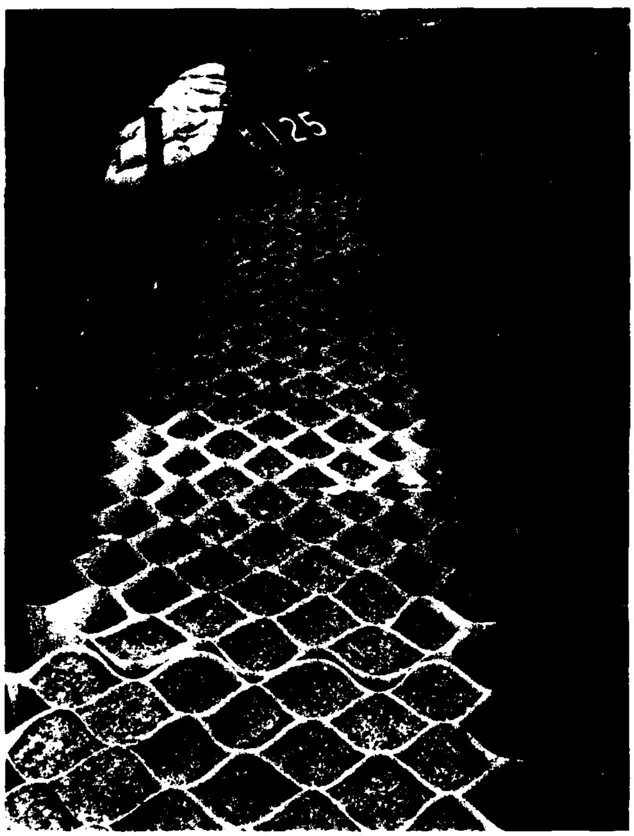

Photo 98. End view of Section 4-2 at the end of the 2.0-ft static differential head

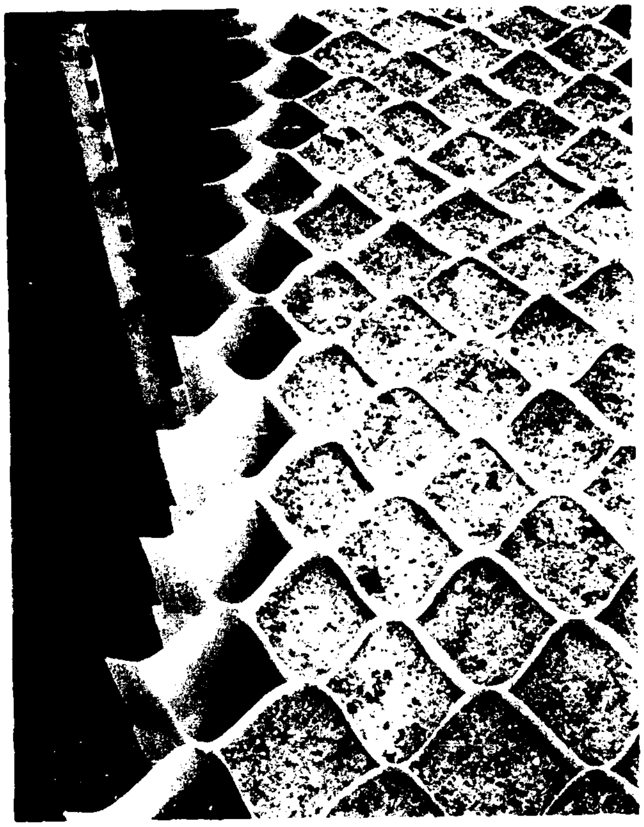

Photo 99. Top view of unfiltered portion of Section 4-2 at the end of the 2.0-ft static differential head

Photo 100. Landside view of Section 4-2 at the end of the 2.0-ft static differential head

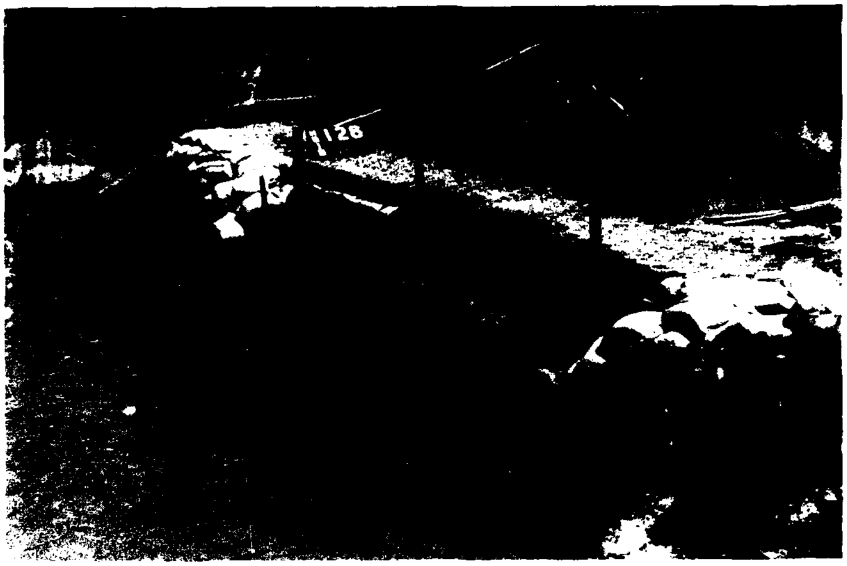

Photo 101. Riverside view of Section 4-2 during wave action at the 1.0-ft water level

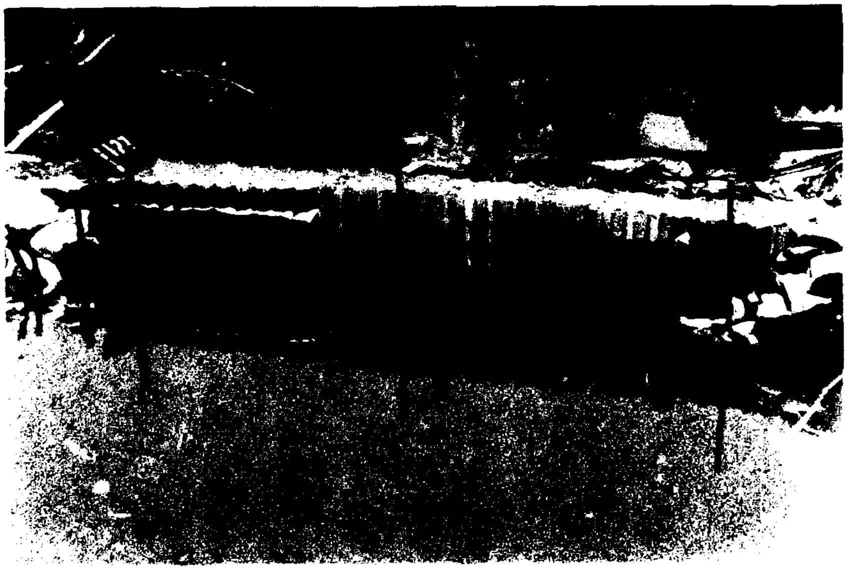

Photo 102. Riverside view of Section 4-2 after 0.5 hr of wave action at the 1.0-ft water level

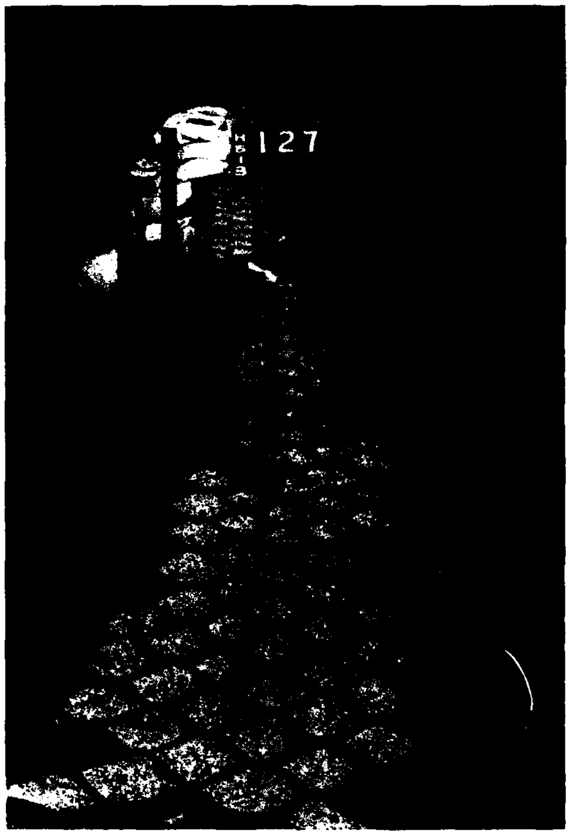

Photo 103. End view *of Section* 4-2
after 0.5 hr of wave action at the
1.0-ft water level

Photo 104. Top, closeup view of riverside cell in
middle of Section 4-2 after 0.5 hr of wave action
at the 1.0-ft water level

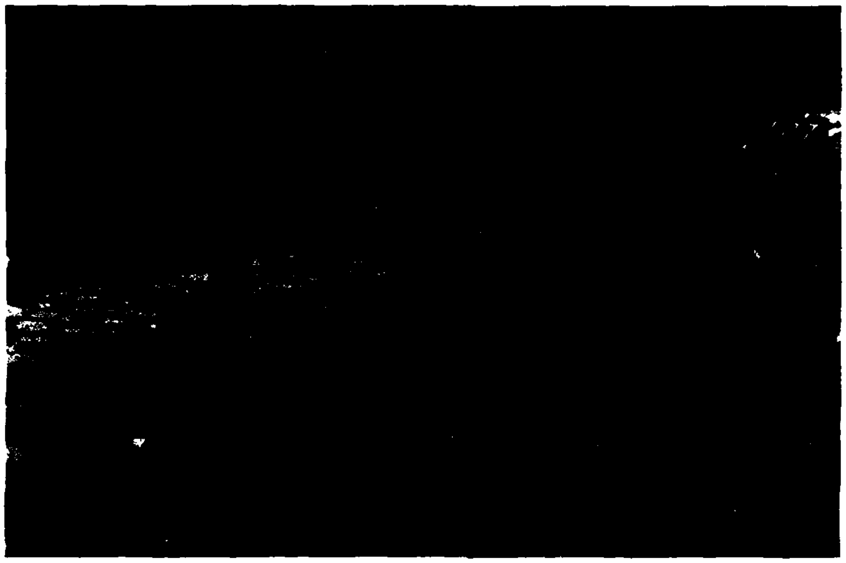

Photo 105. Landside view of Section 4-2 after 0.5 hr of wave action at the 1.0-ft water level

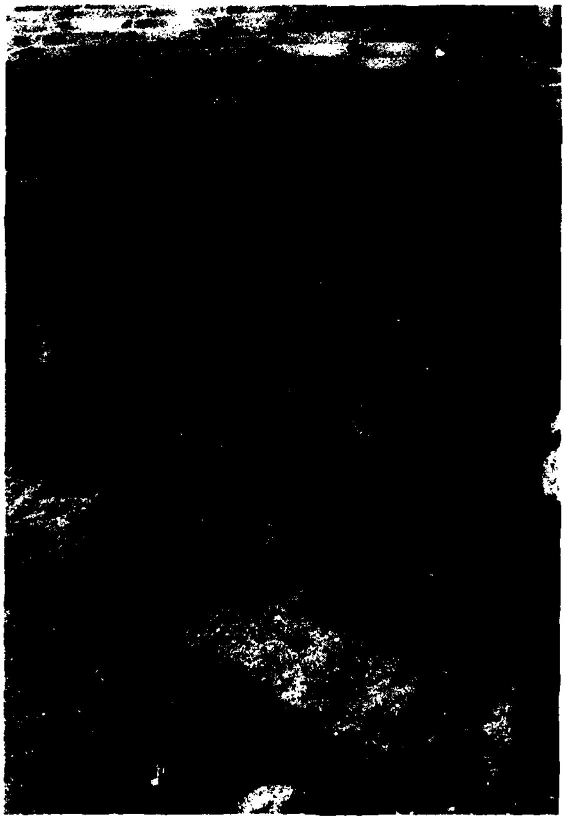

Photo 106. Close-up of cell breakage on landside of Section 4-2 after 0.5 hr of wave action at the 1.0-ft water level

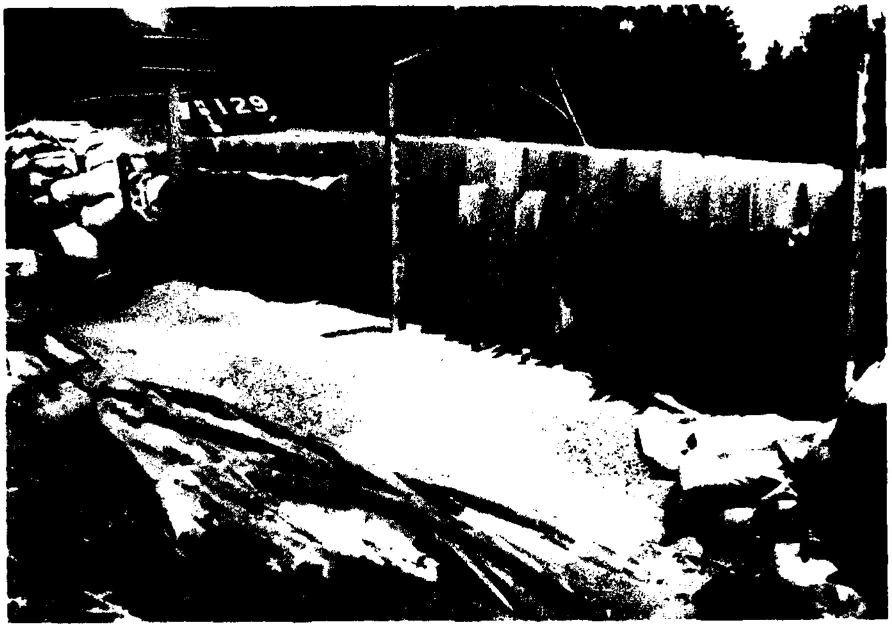

Photo 107. Riverside view of Section 4-2 after 19 hr of wave action at the 1.0-ft water level

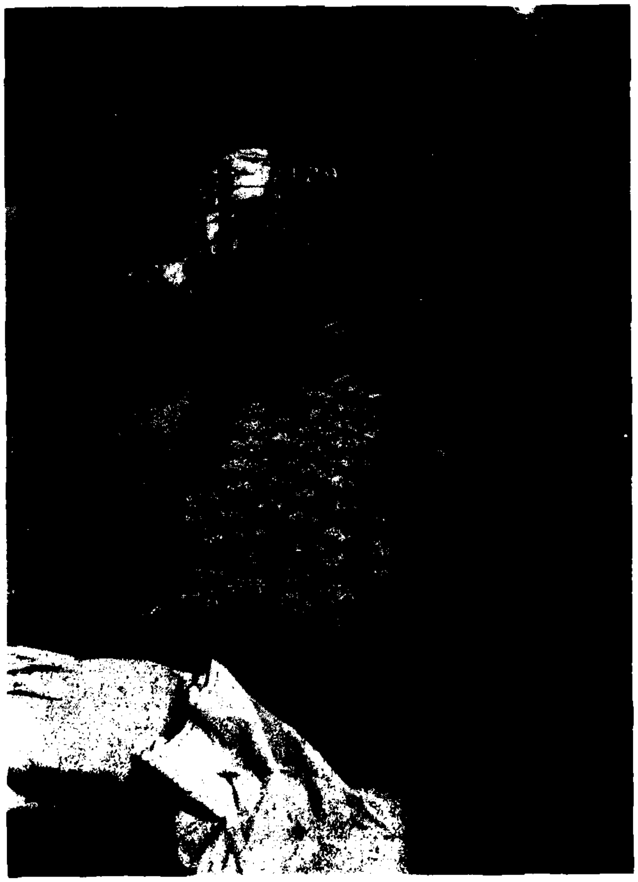

Photo 108. End view of Section 4-2 after 19 hr of wave action at the 1.0-ft water level

Photo 109. Closeup top view of empty cells on the riverside of the burlap filtered portion of Section 4-2 after 19 hr of wave action at the 1.0-ft water level

Photo 110. Closeup top view of empty cells on the unfiltered portion of Section 4-2 after 19 hr of wave action at the 1.0-ft water level

Photo 111. Landside view of Section 4-2 after 19 hr of wave action at the 1.0-ft water level

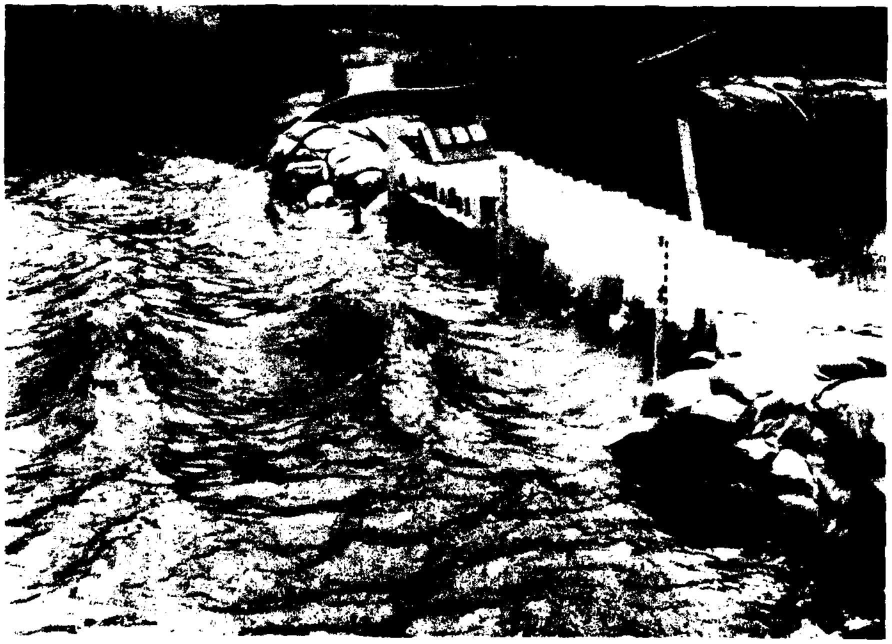

Photo 112. Riverside view of Section 4-2 during wave action at the 2.0-ft water level

Photo 113. Landside view of Section 4-2 showing the seepage that was occurring at the start of wave action at the 2.0-ft water level

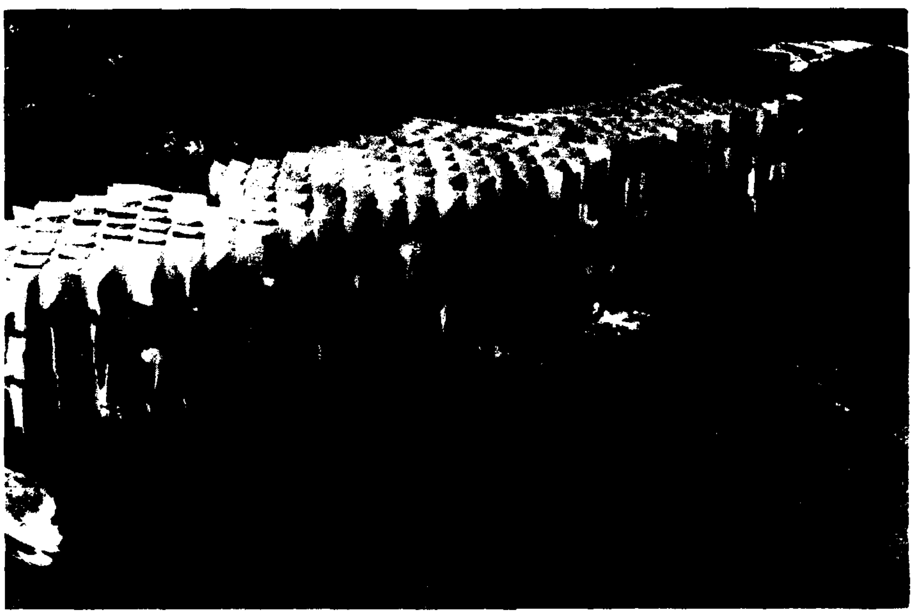

Photo 114. Landside view of Section 4-2 showing the seepage that was occurring after 2.5 hr of wave action at the 2.0-ft water level

Photo 115. Riverside view of Section 4-2 after 2.5 hr of wave action at the 2.0-ft water level

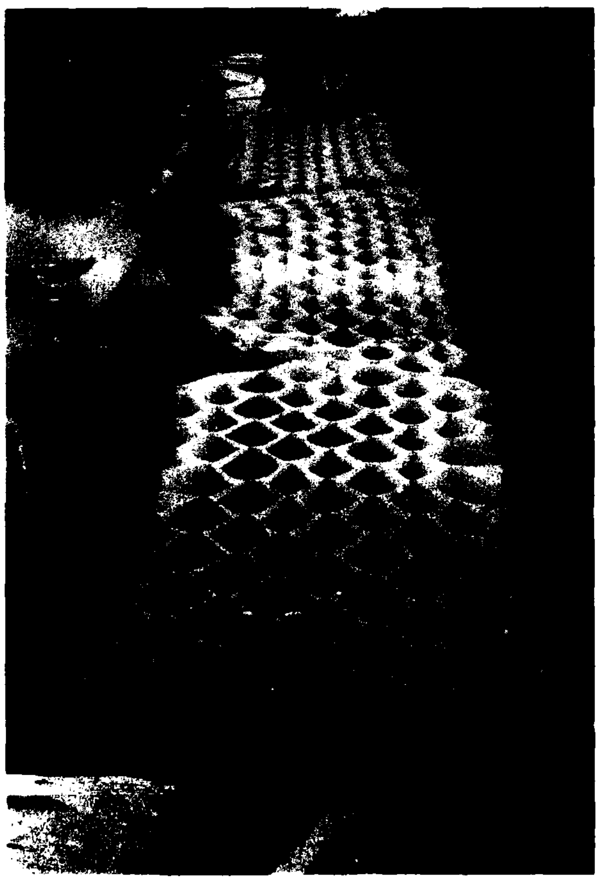

Photo 116. End view of Section 4-2 after 2.5 hr of wave action at the 2.0-ft water level

Photo 117. Landside view of Section 4-2 after 2.5 hr of wave action at the 2.0-ft water level

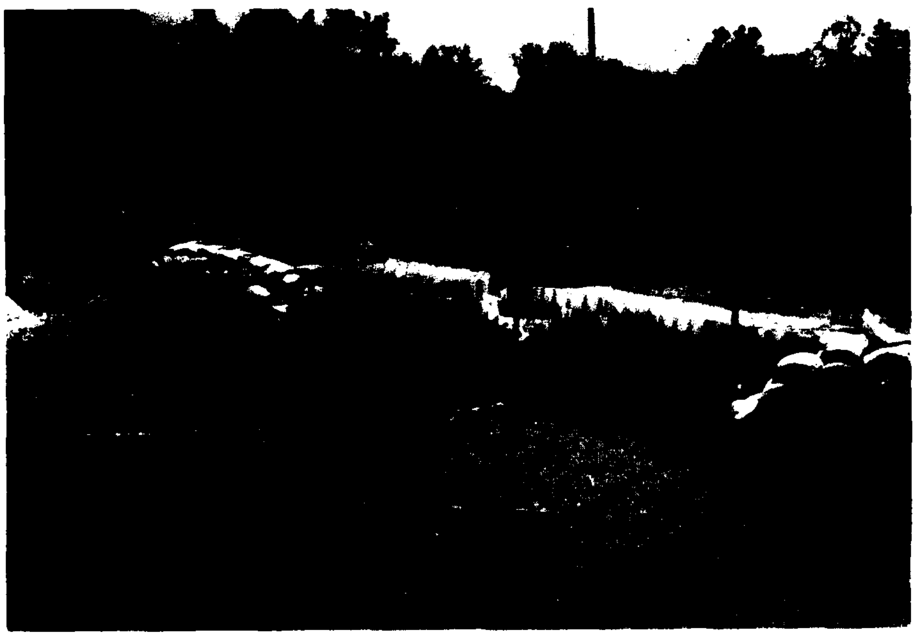

Photo 118. Riverside view of Section 4-2 showing failure of structure occurring while water level is being raised to run wave action at the 3.0-ft water level

Photo 119. Landside view of Section 4-2 showing failure of structure occurring while water level is being raised to run wave action at the 3.0-ft water level

Photo 120. Riverside view of Section 4-2 at end of entire test

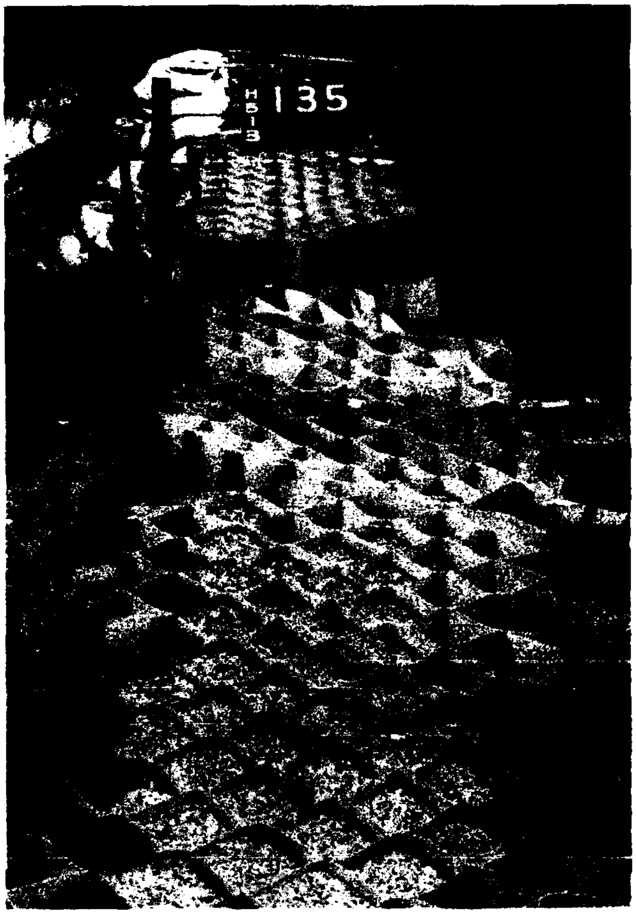

Photo 121. End view of Section 4-2 at end of entire test

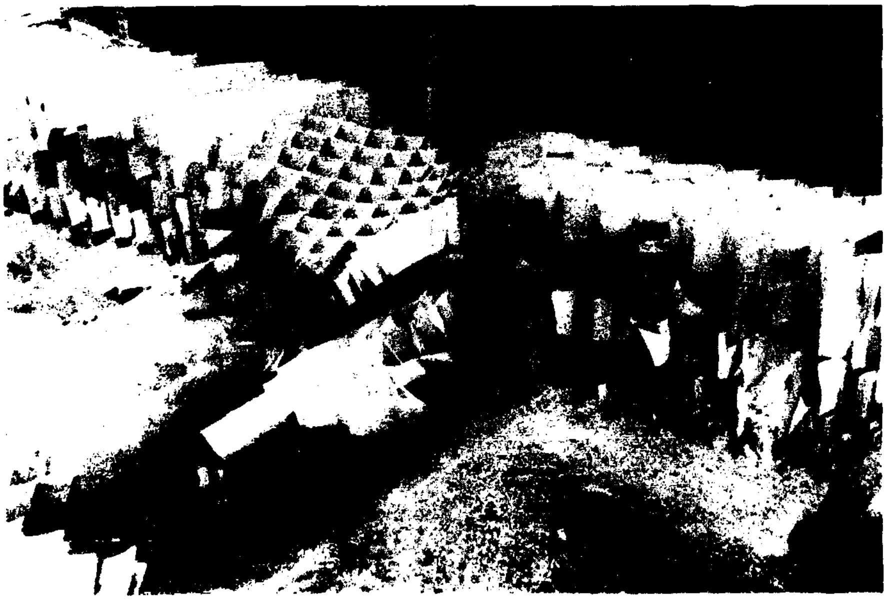

Photo 122. Landside view of Section 4-2 at end of entire test

Photo 123. Riverside view of Section 4-3 before testing

Photo 124. End view of Section 4-3 before testing

Photo 125. Cutting trench during construction of Section 4-3

Photo 126. Installation of posts during construction of Section 4-3

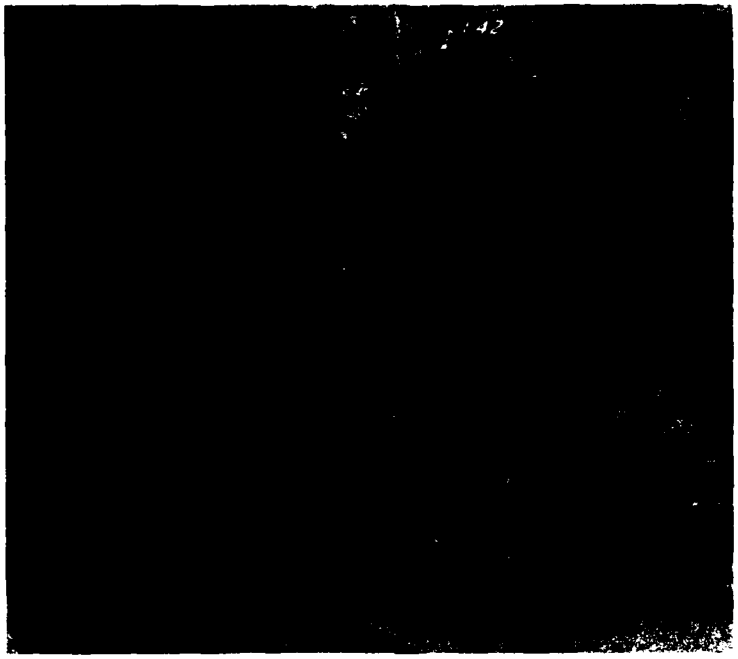

Photo 127. Landside view of Section 4-3 showing seepage at the start of the 1.0-ft static differential head

Photo 128. Landside view of Section 4-3 showing flow under Section 4-3 just prior to stopping the 1.0-ft static differential head test

Photo 129. Riverside view of Section 4-3 after the 1.0-ft static differential head

Photo 130. End view of Section 4-3 after the 1.0-ft static differential head

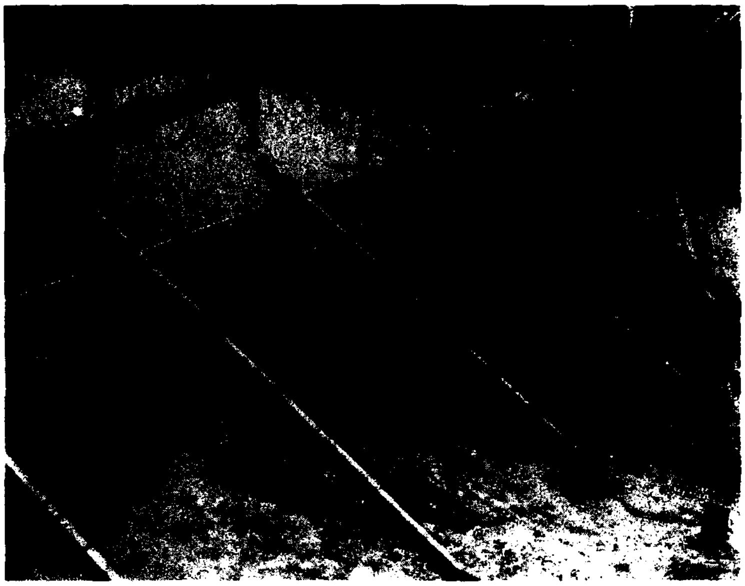

Photo 131. Landside view of Section 4-3 after the 1.0-ft static differential head

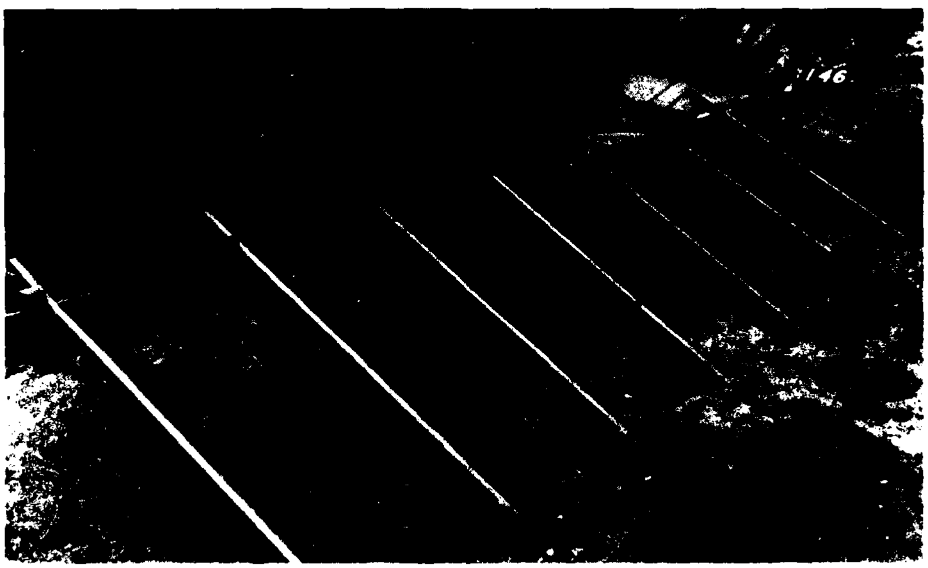

Photo 132. Landside view of Section 4-3-A before testing

Photo 133. Riverside view of Section 4-3-A at the end of the static differential head tests

Photo 134. Landside view of Section 4-3-A at the end of the static differential head tests

Photo 135. Riverside view of Section 4-3-A during wave action at the 1.0-ft water level

Photo 136. Riverside view of Section 4-3-A during wave action at the 2.0-ft water level

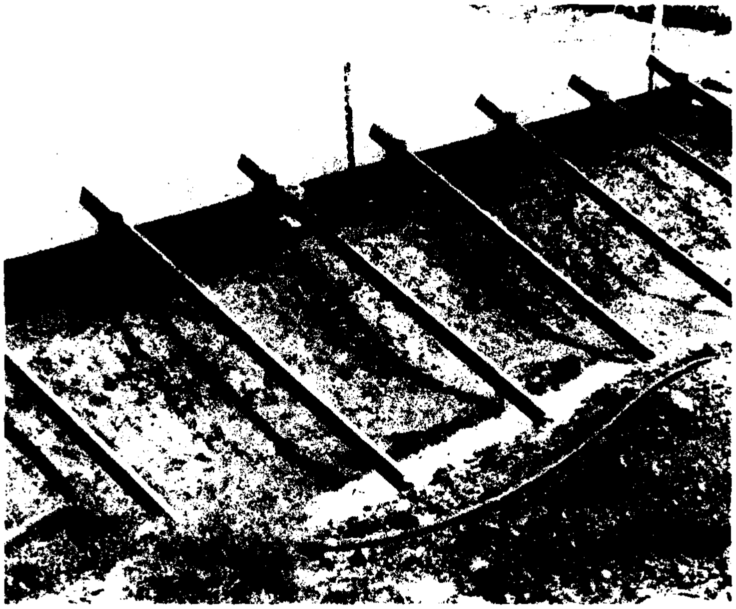

Photo 137. Landside view of Section 4-3-A during wave action at the 2.0-ft water level

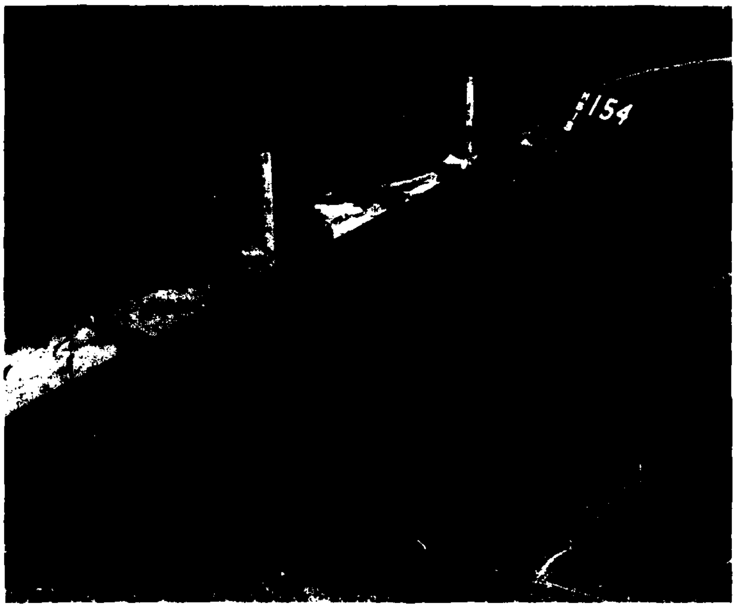

Photo 138. Landside view of Section 4-3-A after tests had been inactive for 2 months

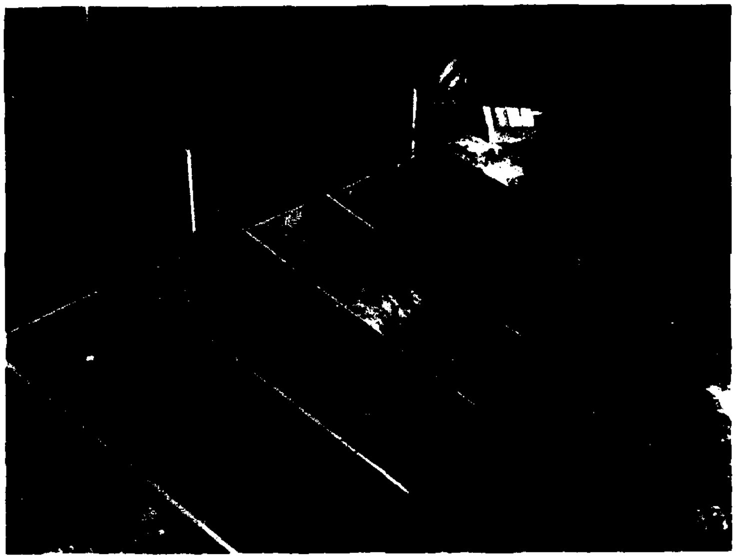

Photo 139. Landside view of Section 4-3-A after grass removal

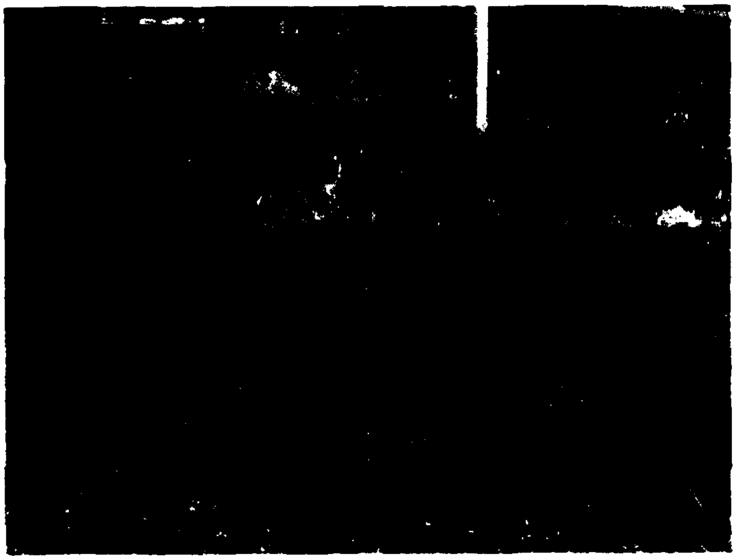

Photo 140. Landside view of Section 4-3-A showing wave overtopping at the 2.5-ft testing depth

Photo 141. Landside view of Section 4-3-A after 16 hr of wave attack at the 2.5-ft water level (end of entire test)

Photo 142. Close-up of Section 4-3-A showing the erosion produced by overtopping waves at the 2.5-ft water level

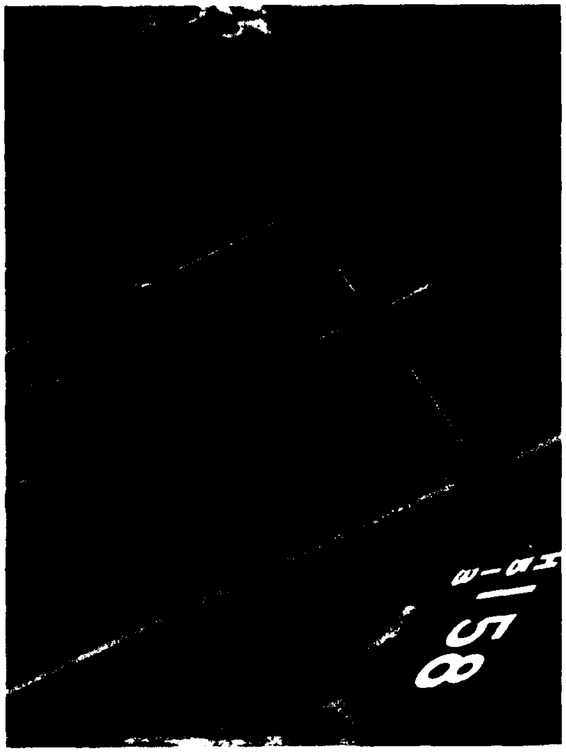

Photo 143. Landside view of
Section 4-3-B before testing

Photo 144. Landside view of Section 4-3-B at the end
of the 1.5-ft static differential head test

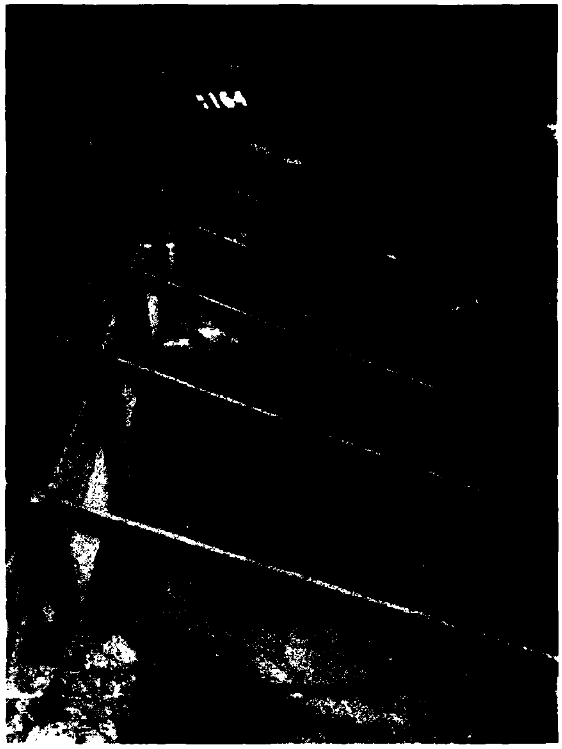

Photo 145. Landside view of Section 4-3-B at the end of the 2.0-ft static differential head test

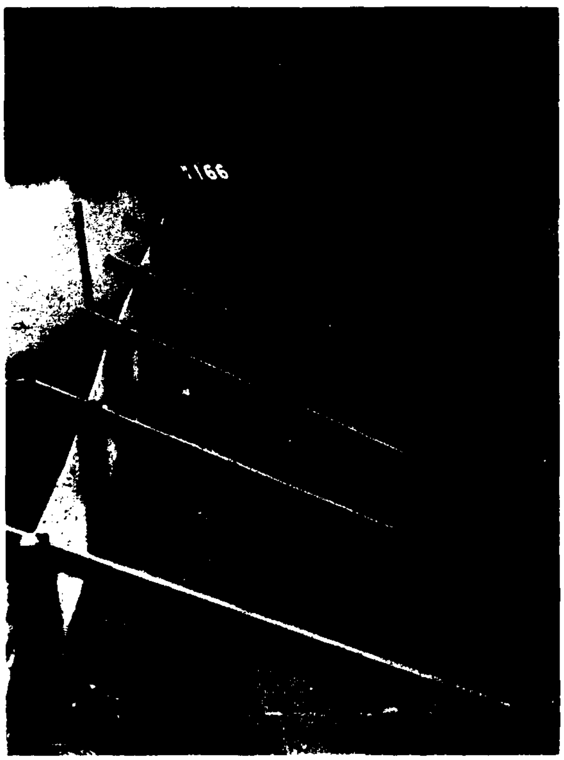

Photo 146. Landside view of Section 4-3-B at the end of the 2.5-ft static differential head test

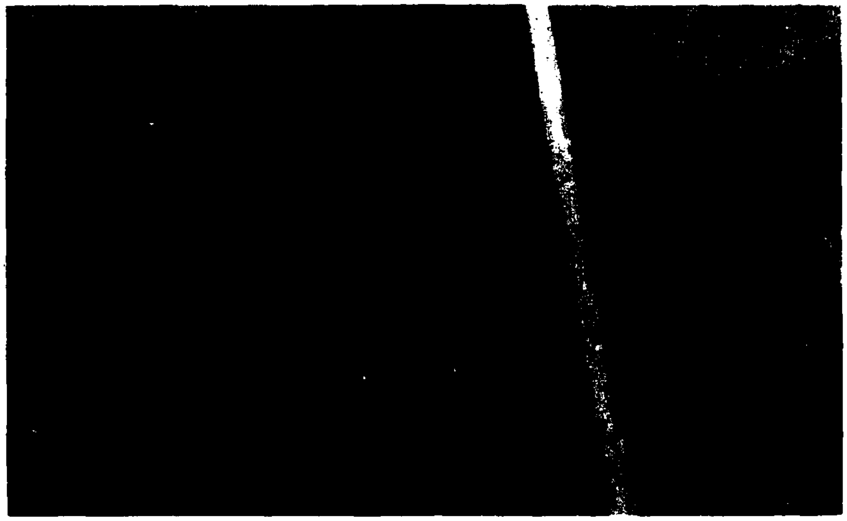

Photo 147. Landside closeup of Section 4-3-B showing seepage around support post and surface erosion at the end of the 2.5-ft static head test

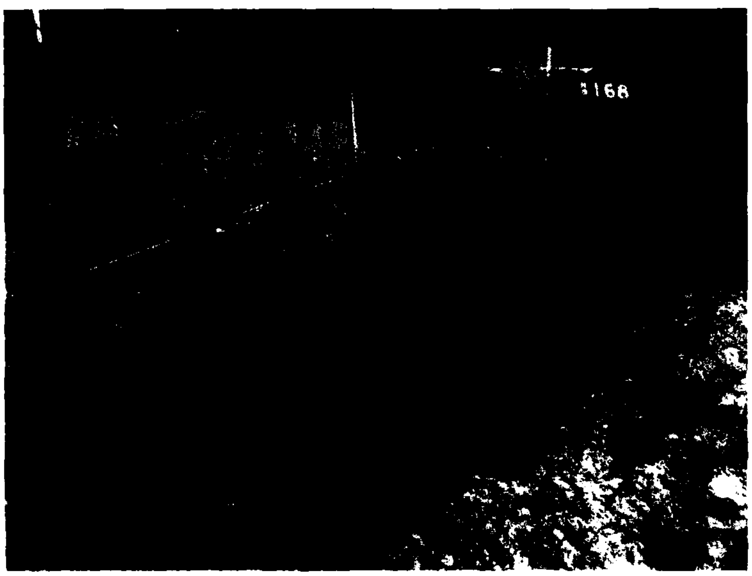

Photo 148. Landside view of Section 4-3-B after failure during the 2.9-ft static differential head test (end of entire test)

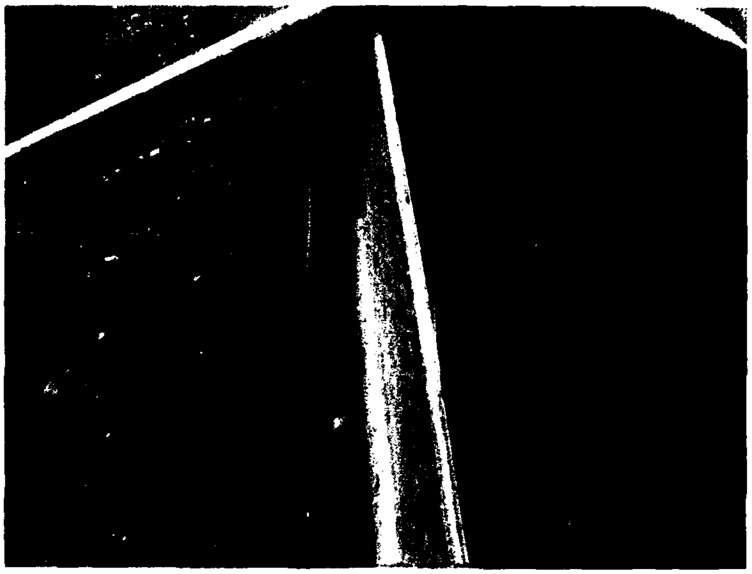

Photo 149. Landside close-up of Section 4-3-B showing points of failure

Photo 150. Riverside close-up of Section 4-3-B showing point of failure

Photo 151. Landside view of Section 4-3-C
before testing

Photo 152. Landside view of Section 4-3-C at the
end of the 1.0-ft static differential head test

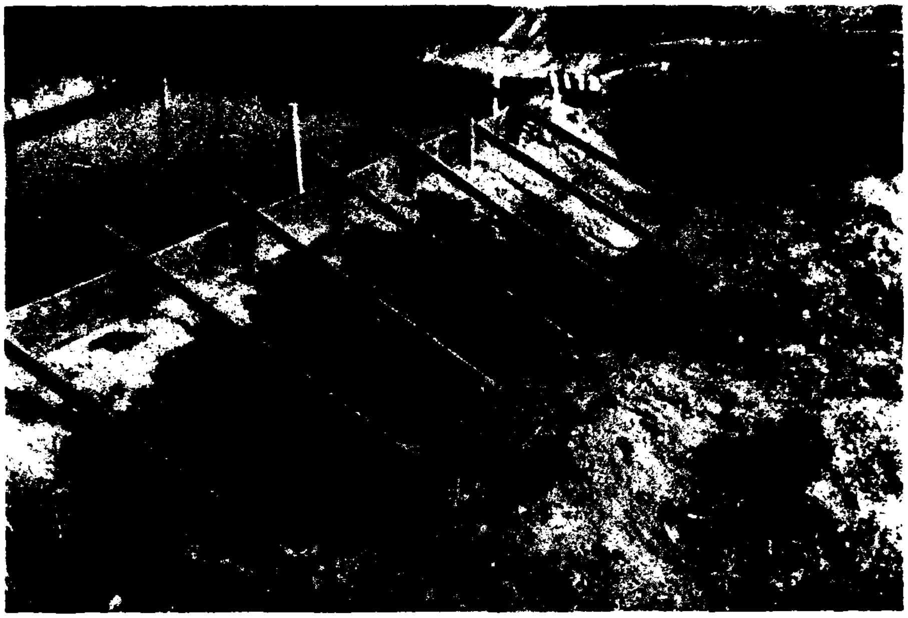

Photo 153. Landside view of Section 4-3-C after 24 hr of the 1.5-ft static differential head test

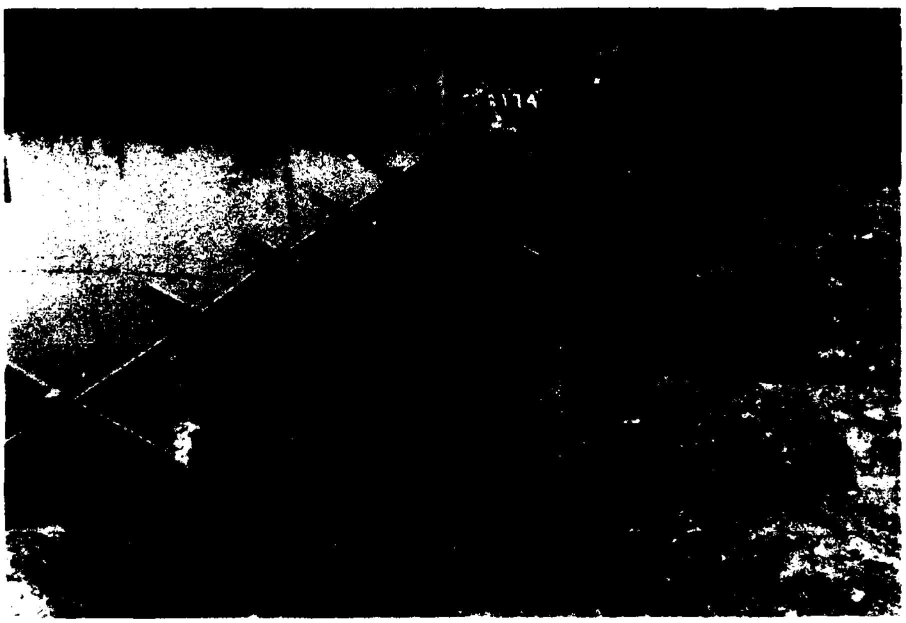

Photo 154. Landside view of Section 4-3-C at the end of the 1.5-ft static differential head test

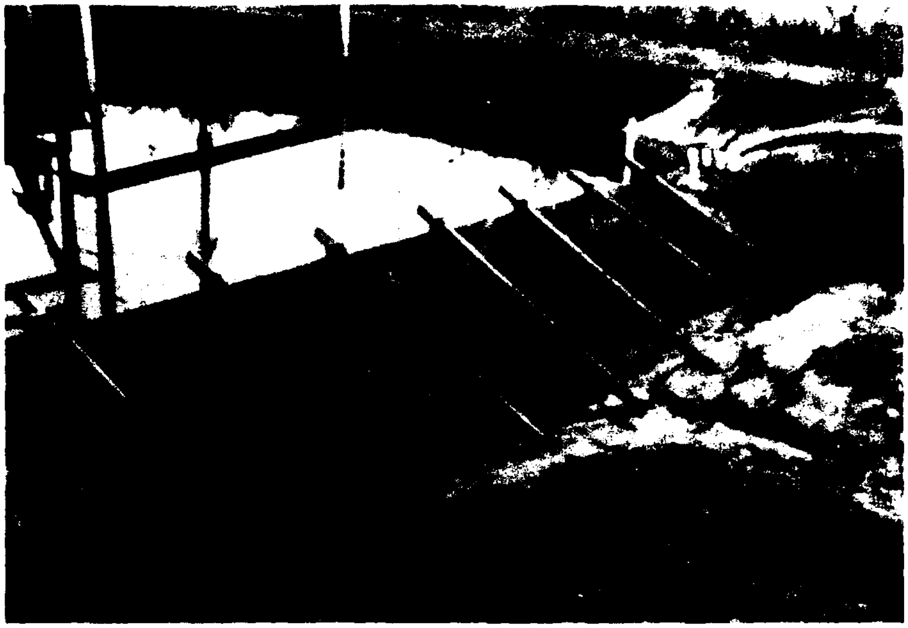

Photo 155. Landside view of Section 4-3-C at the end of the 2.0-ft static differential head test

Photo 156. Landside close-up showing seepage that was occurring at the end of the 2.0-ft static differential head test

Photo 157. Landside close-up showing seepage that was occurring during the 2.5-ft static differential head test

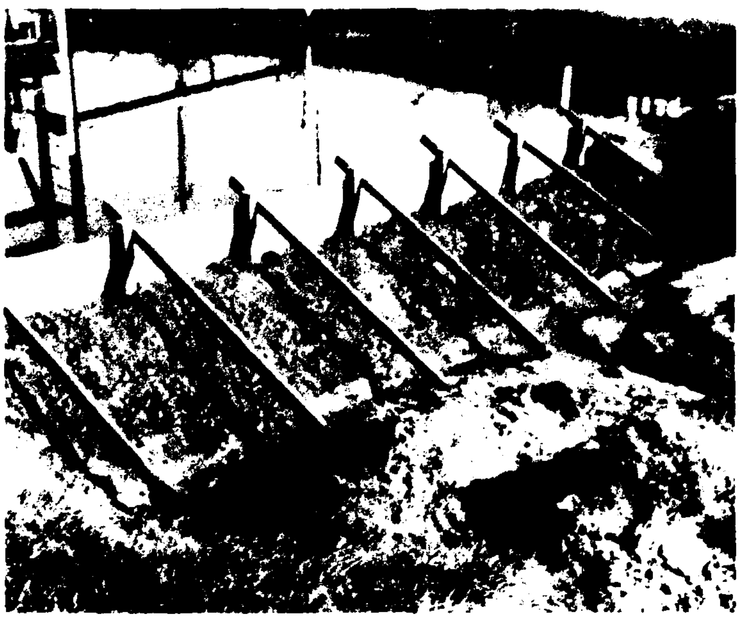

Photo 158. Landside view of Section 4-3-C at the end of the 2.5-ft static differential head test

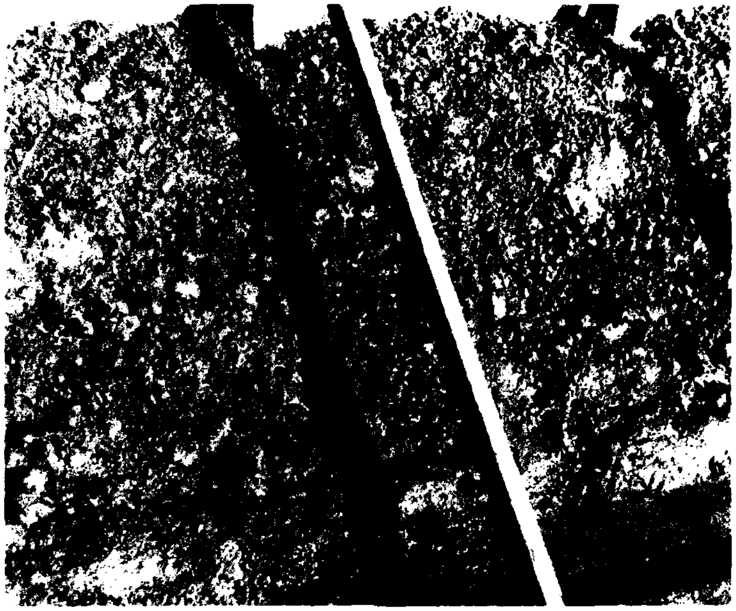

Photo 159. Landside close-up of Section 4-3-C showing seepage around support post and surface erosion produced by the 2.5-ft static differential head

Photo 160. Landside view of Section 4-3-C during failure

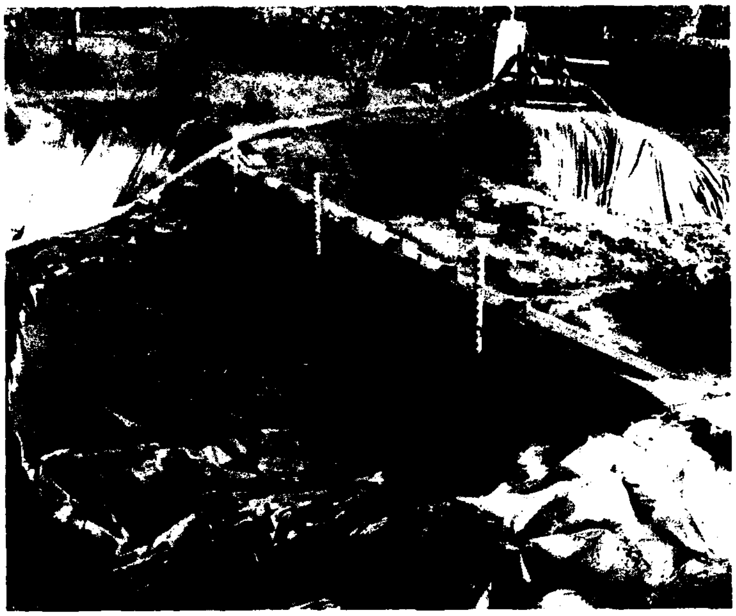

Photo 161. Riverside view of Section 4-3-C at end of test

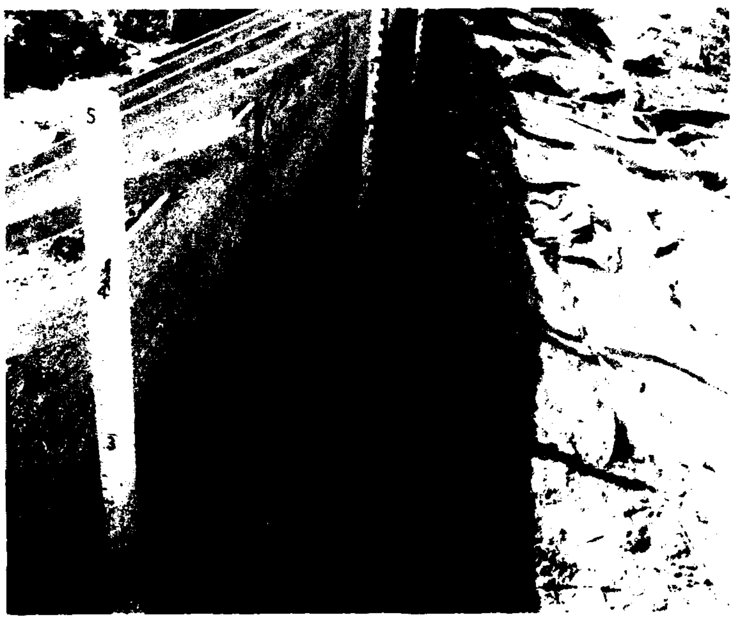

Photo 162. Riverside close-up showing failure point of Section 4-3-C

Photo 163. Landside view showing eroded earth backing of Section 4-3-C

Photo 164. Landside close-up showing failure point of Section 4-3-C

Photo 165. Riverside view of Section 4-3-D during spraying of asphalt sealer

Photo 166. Riverside view of Section 4-3-D after rains had washed away uncured asphalt

Photo 167. Landside view of Section 4-3-D after rains had washed away uncured asphalt

Photo 168. Riverside view of cured asphalt on Section 4-3-D

Photo 169. Landside view of cured asphalt on Section 4-3-D

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Photo 171. Landside view of Section 4-3-D at the end of the 1.5-ft static differential head test

Photo 172. Landside close-up of Section 4-3-D showing bulge in asphalt at the end of the 2.0-ft static differential head test

Photo 173. Landside view of Section 4-3-D during failure at the 2.5-ft static differential head

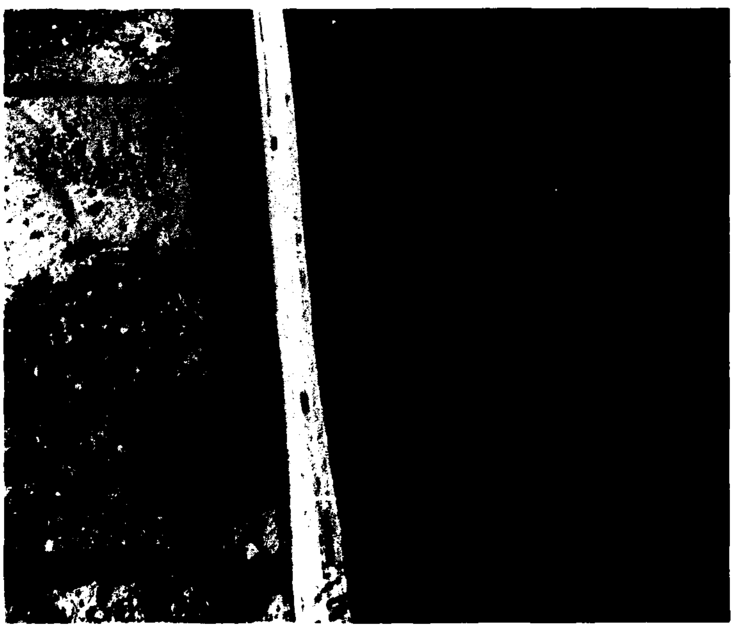

Photo 174. Close-up showing failure point of Section 4-3-D

Photo 175. Riverside view of Section 4-4 before testing

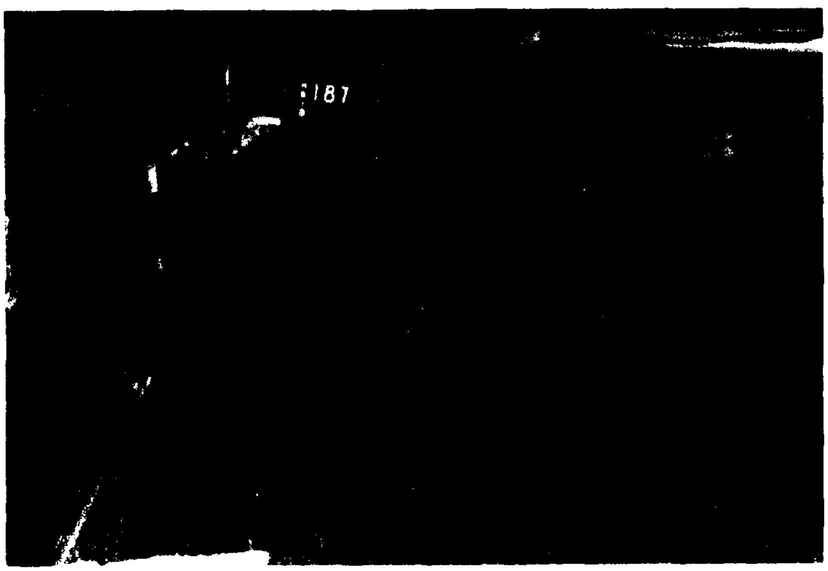

Photo 176. Landside view of Section 4-4 before testing

Photo 177. Placement of support posts during construction of Section 4-4

Photo 178. Riverside view of completed flashboard of Section 4-4

Photo 179. Landside view of Section 4-4 after scarification of existing levee and prior to construction of earth backing

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Photo 182. Section 4-4 during wave attack at the 1.0-ft water level

Photo 183. End view of Section 4-4 after 19 hr of wave attack at the 1.0-ft water level

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Photo 189. Landside view of Section 4-4 after approximately 0.5 hr of wave attack at the 2.5-ft water level

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Photo 191. Landside view of Section 4-4 after approximately 1.0 hr of wave attack at the 2.5-ft water level

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Photo 194. Landside view of boil that formed after
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Photo 195. Riverside view of Section 4-4 at end of test

Photo 196. Riverside view showing failure point of Section 4-4

Photo 197. End view of Section 4-4 at end of test

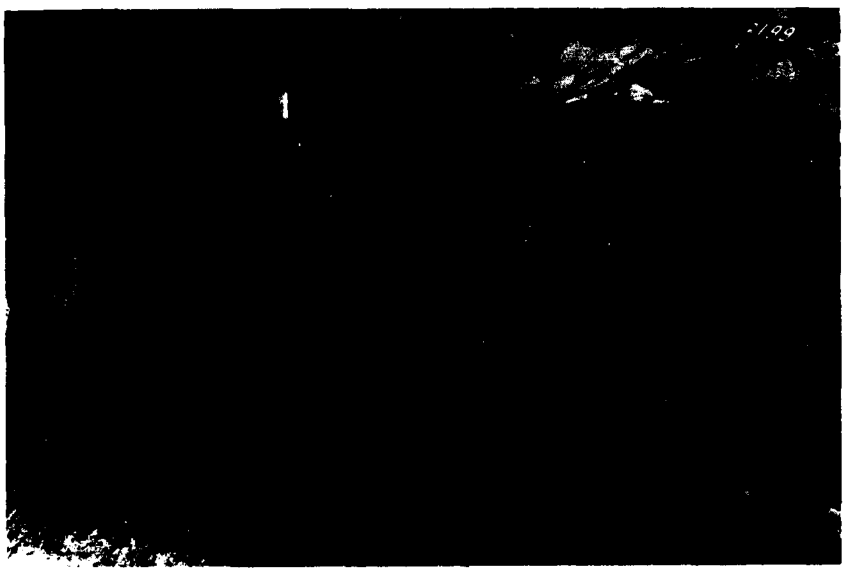

Photo 198. Landside view of Section 4-4 at end of test

Photo 199. Riverside view of Section 4-5 before testing

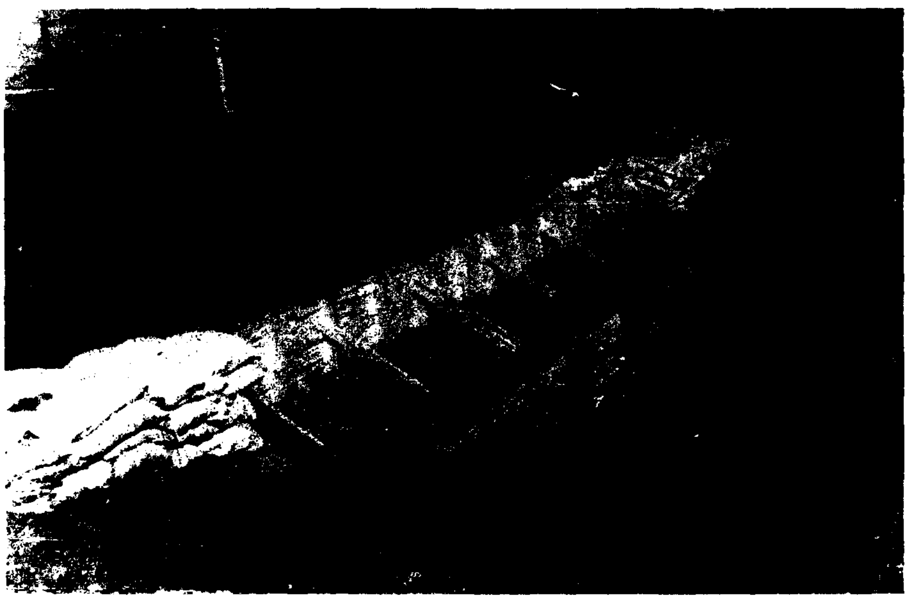

Photo 200. Landside view of Section 4-5 before testing

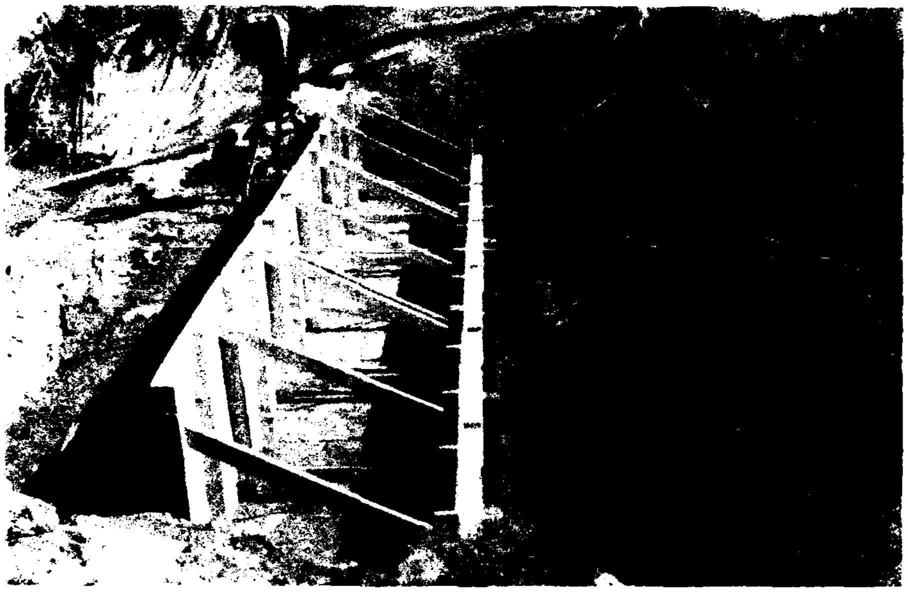

Photo 201. Side view of Section 4-5 showing completed mud boxes prior to placing fill

Photo 202. Side view of Section 4-5 during placement of fill

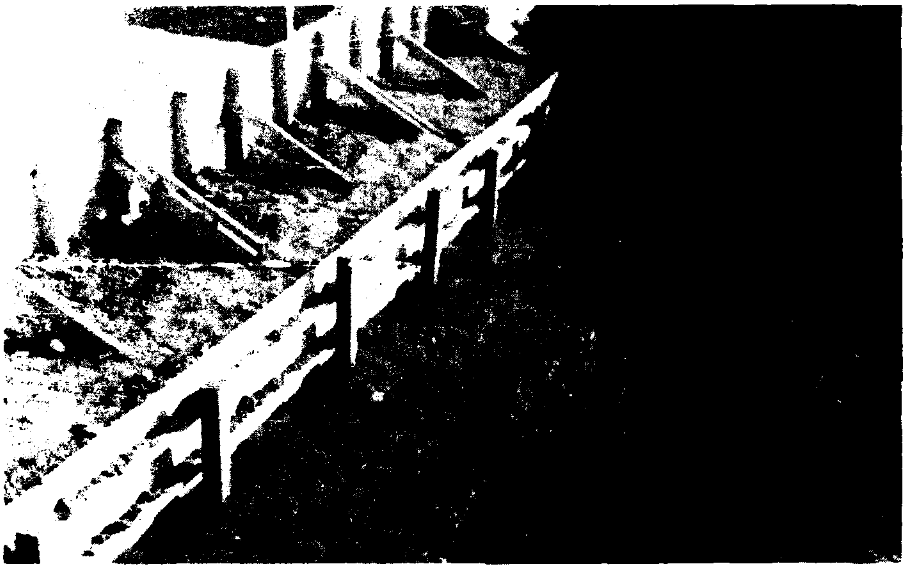

Photo 203. Landside view of Section 4-5 after it failed to retain 1.0-ft static differential head

Photo 204. Riverside view of sandbag repair on Section 4-5

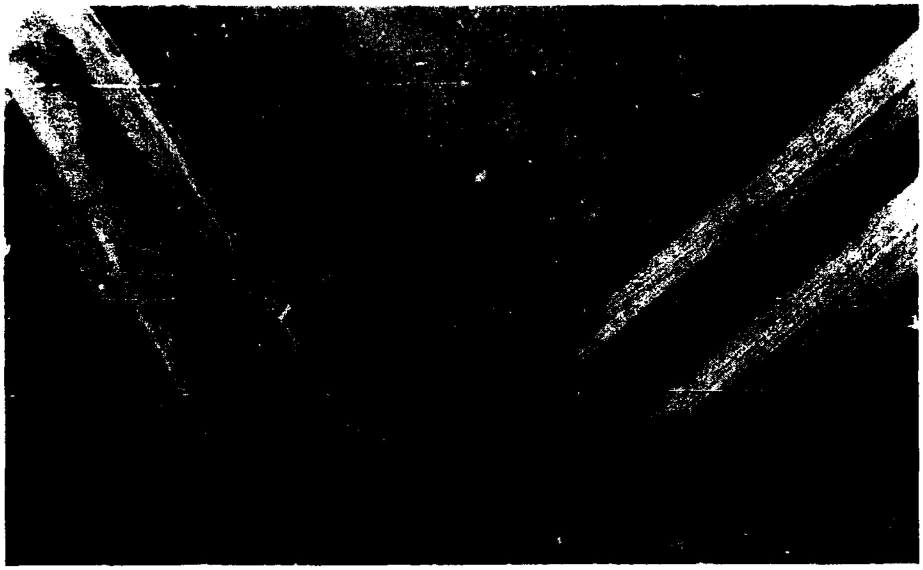

Photo 205. Close-up of earth fill consolidation on Section 4-5

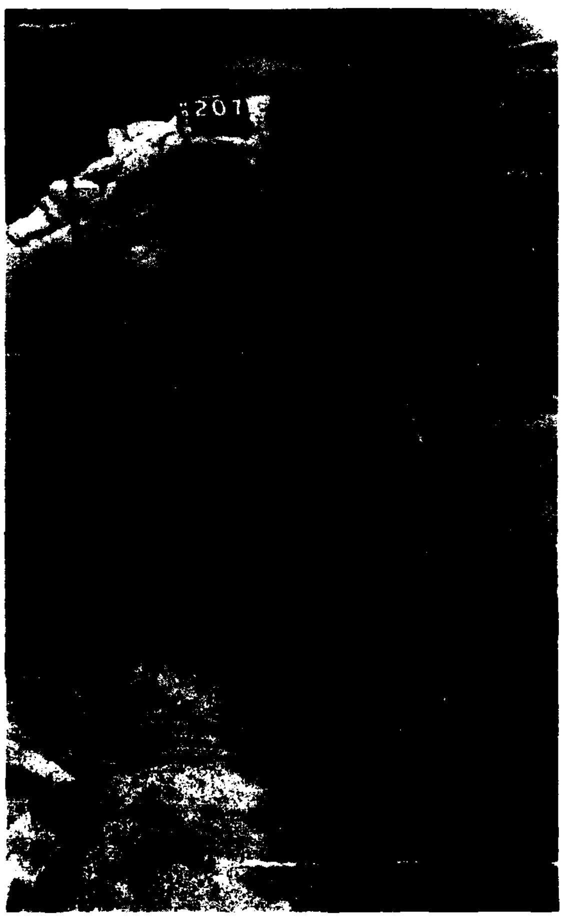

Photo 206. Side view of Section 4-5 after compaction of earth fill

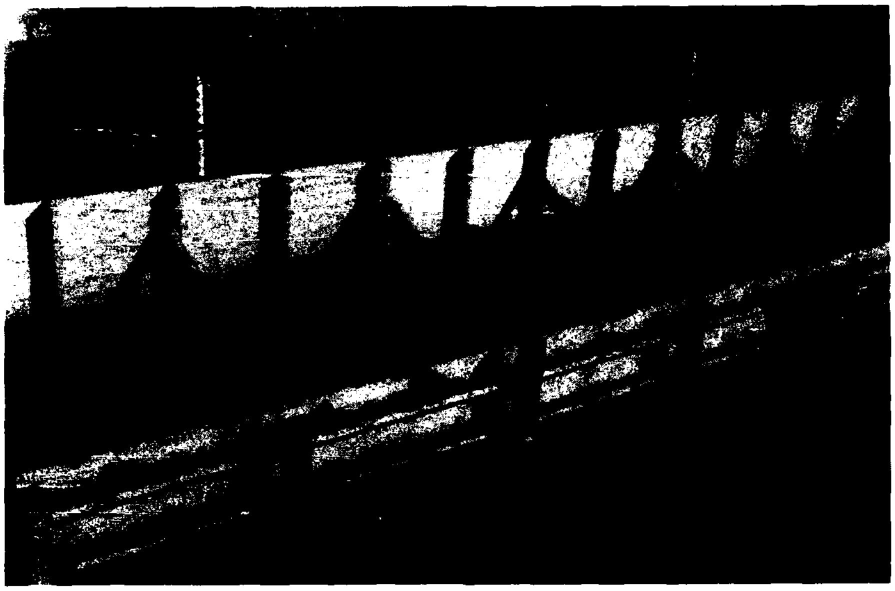

Photo 207. Landside view of Section 4-5 after repair of mud box fill

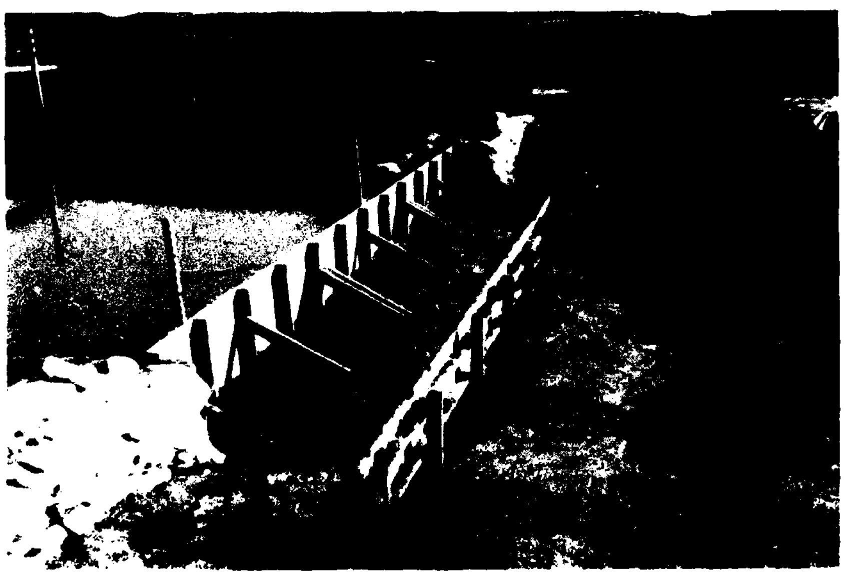

Photo 208. Landside view of Section 4-5 at the end of the 2.0-ft static differential head test

Photo 209. Landside close-up showing water flow under Section 4-5 during the 2.5-ft static differential head test

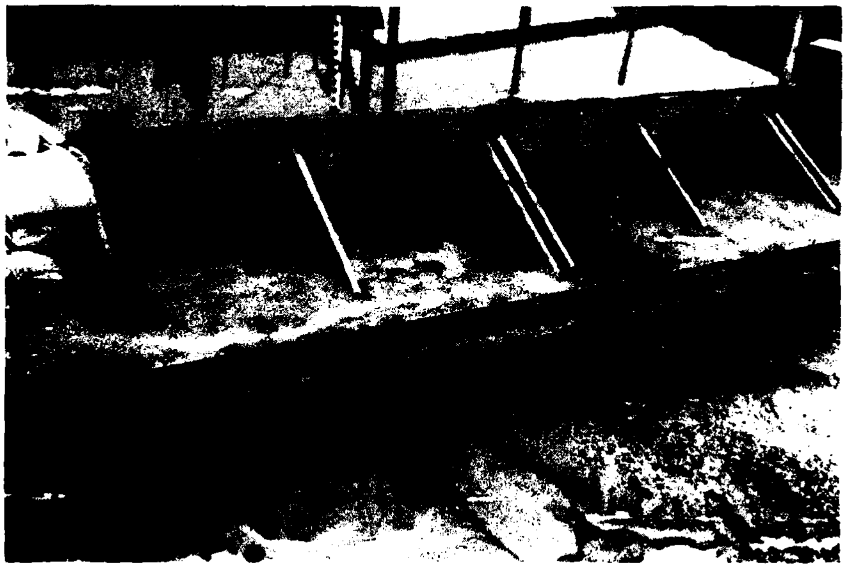

Photo 210. Landside view of Section 4-5 at the end of the 2.5-ft static differential head test

Photo 211. Side view of Section 4-5 after repair of earth fill prior to wave action tests

Photo 212. Riverside view of Section 4-5 at end of test

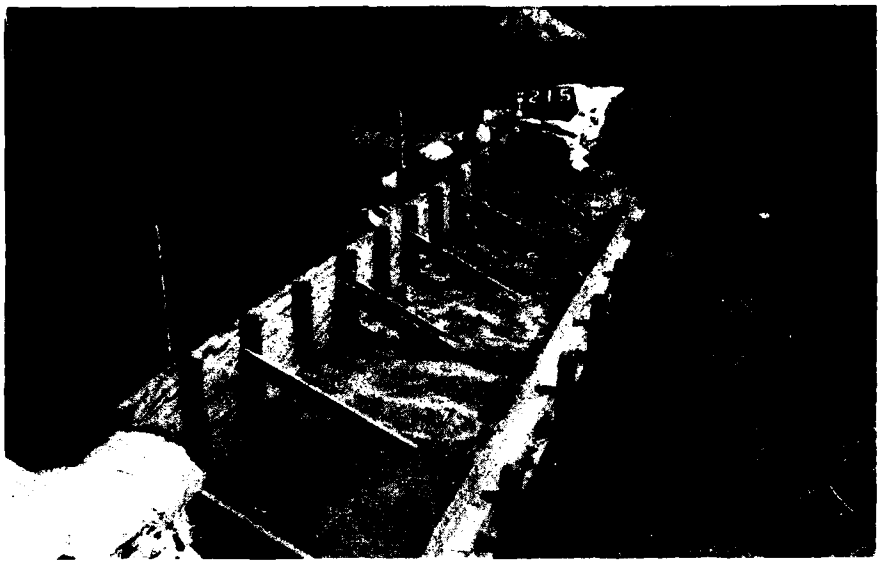

Photo 213. Landside view of Section 4-5 at end of test

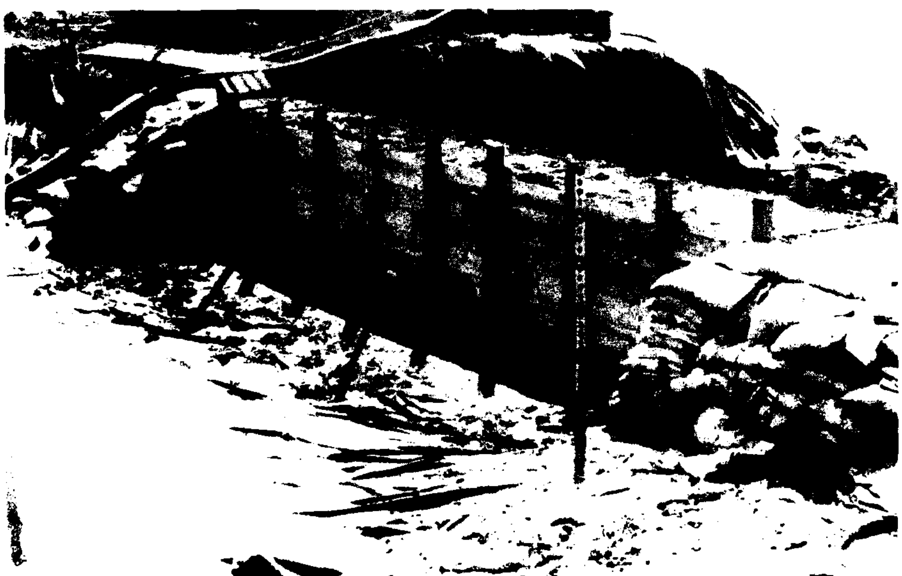

Photo 214. Riverside view of Section 4-6 before testing

Photo 215. Side view of Section 4-6
before testing

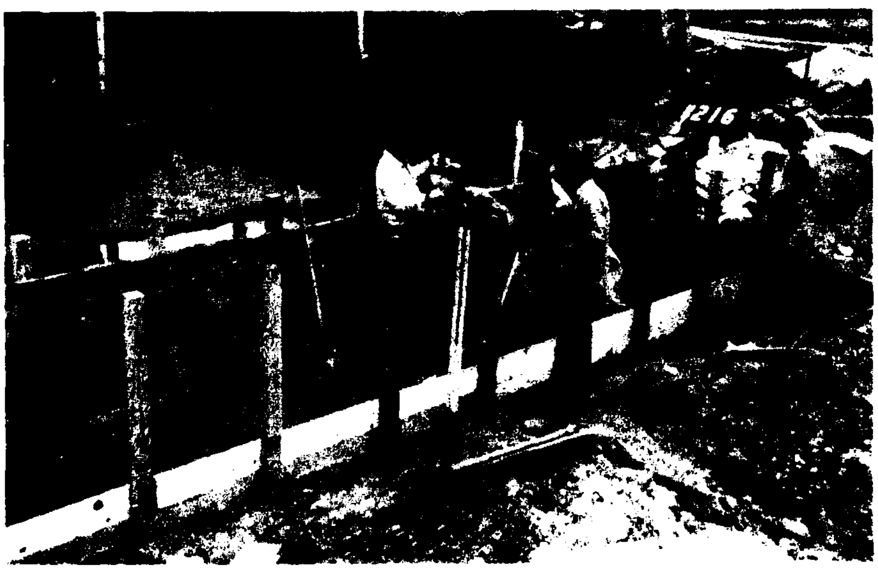

Photo 216. Landside view of Section 4-6 during
construction of plank walls

Photo 217. Side view of Section 4-6 during placement of wire ties

Photo 218. Landside view of Section 4-6 during placement of tamped earth fill

Photo 219. Riverside view of Section 4-6 at the end of the 3.5-ft static differential head test

Photo 220. Landside view of Section 4-6 at the end of the 3.5-ft static differential head test

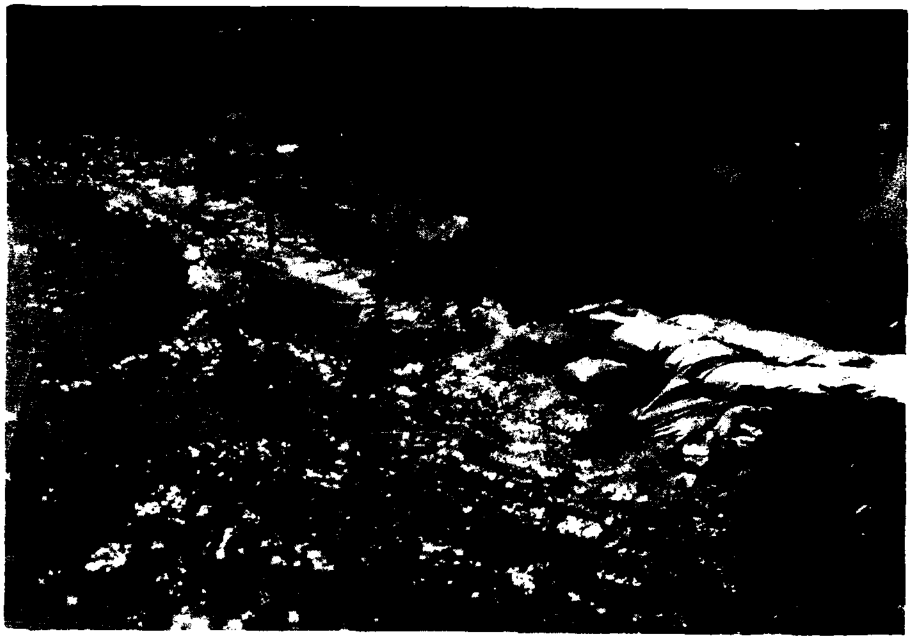

Photo 221. Riverside view of wave action on Section 4-6 at the 3.0-ft water level

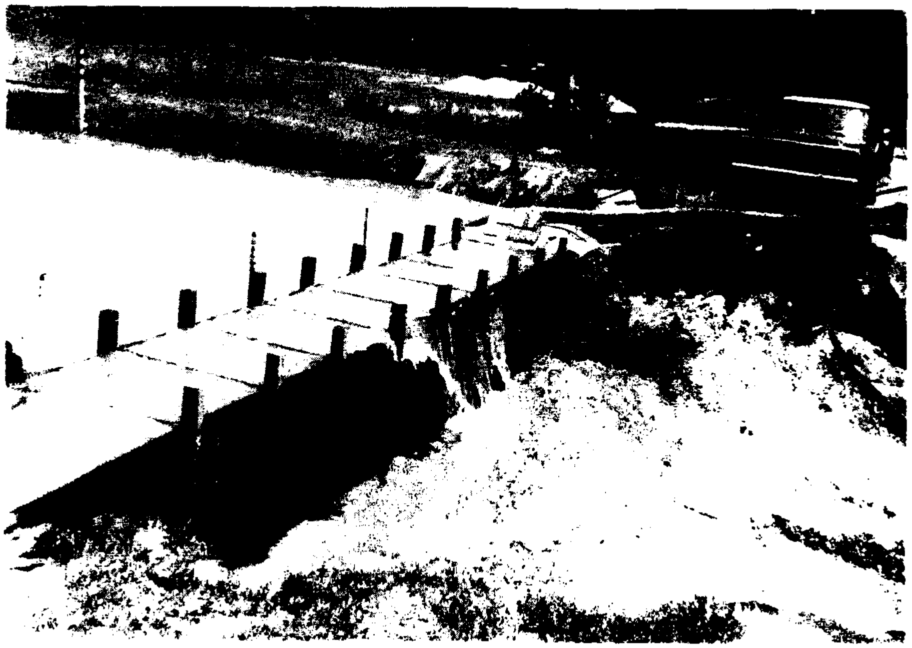

Photo 222. Landside view of wave action on Section 4-6 at the 3.0-ft water level

Photo 223. Riverside view of Section 4-6 at the end of the wave action tests

Photo 224. Side view of Section 4-6 at the end of the wave action tests

Photo 225. Landside view of Section 4-6 at the end of the wave action tests

Photo 226. Riverside view of Section 4-6-A before testing

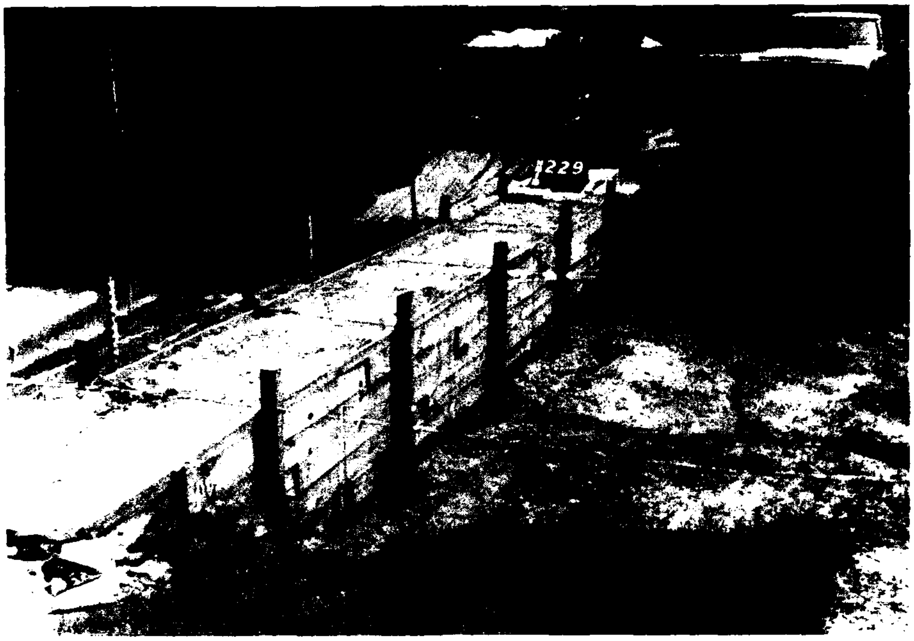

Photo 227. Landside view of Section 4-6-A before testing

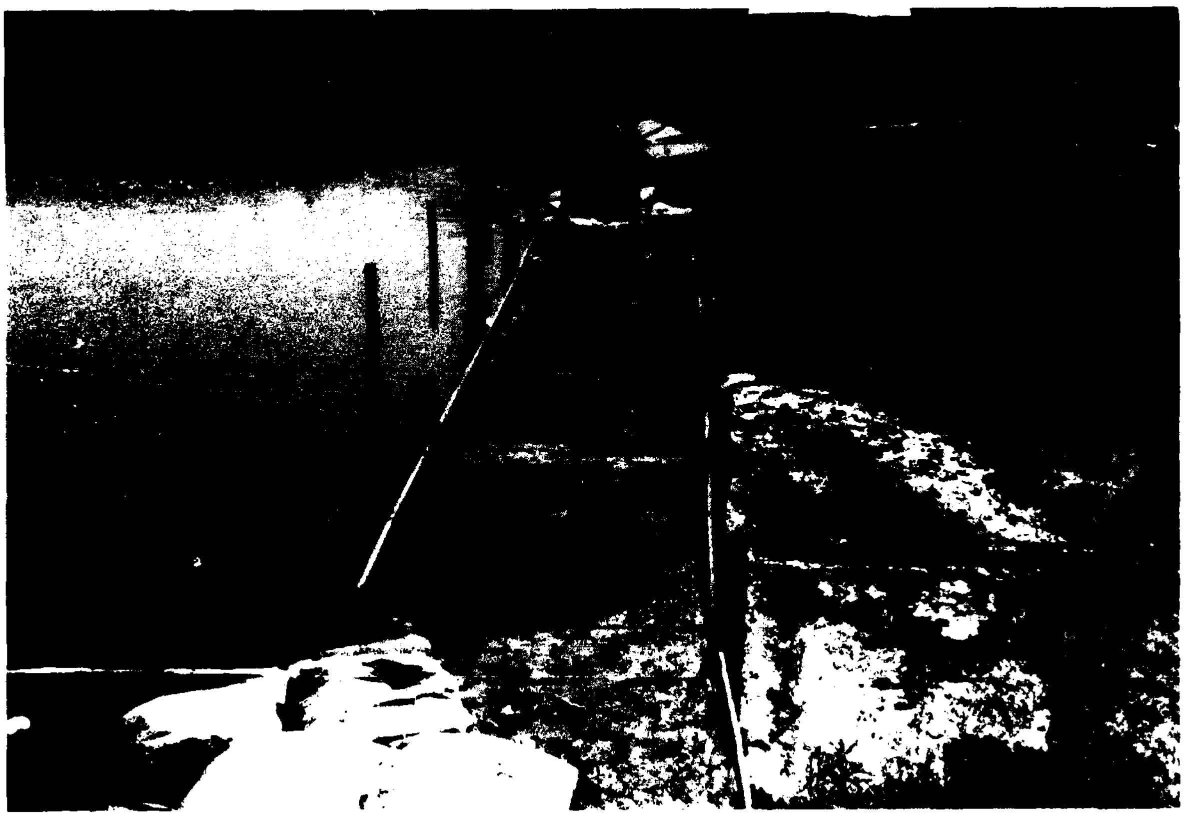

Photo 228. Side view of Section 4-6-A at the end of the 3.5-ft static differential head test

Photo 229. Landside view of Section 4-6-A during wave action at the 3.0-ft water level

Photo 230. Side view of Section 4-6-A at the end of the wave action tests

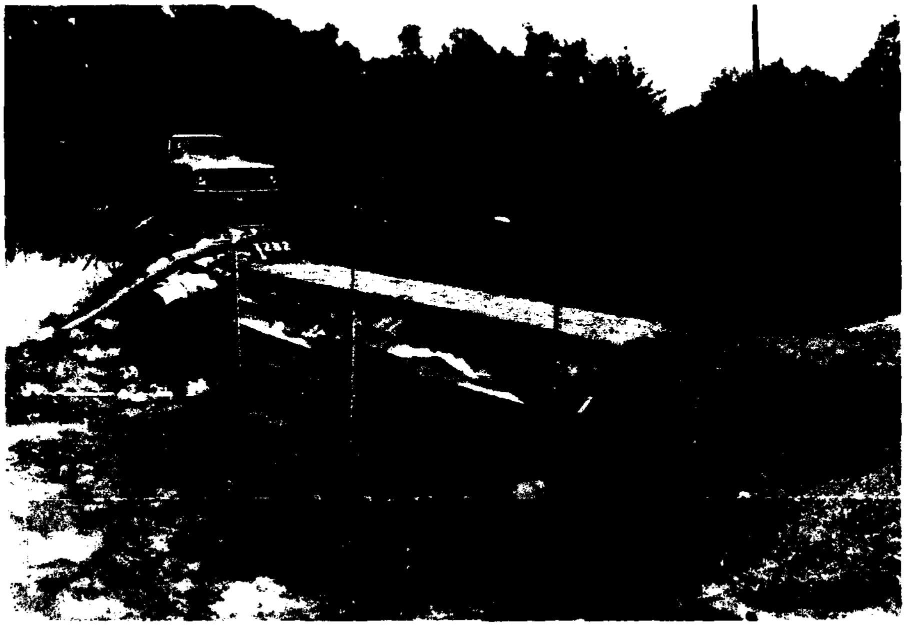

Photo 231. Riverside view of Section 6-1 before testing

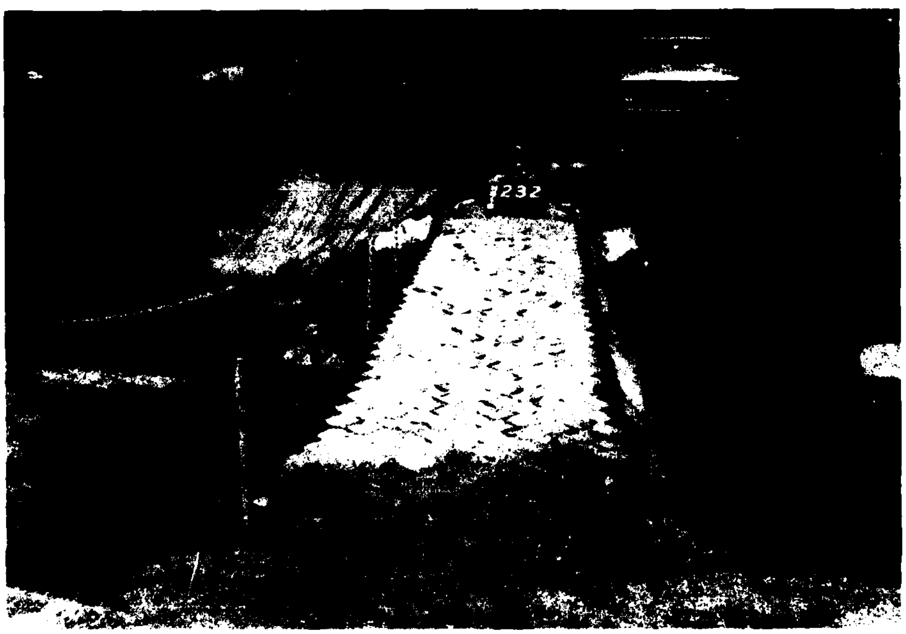

Photo 232. End view of Section 6-1 before testing

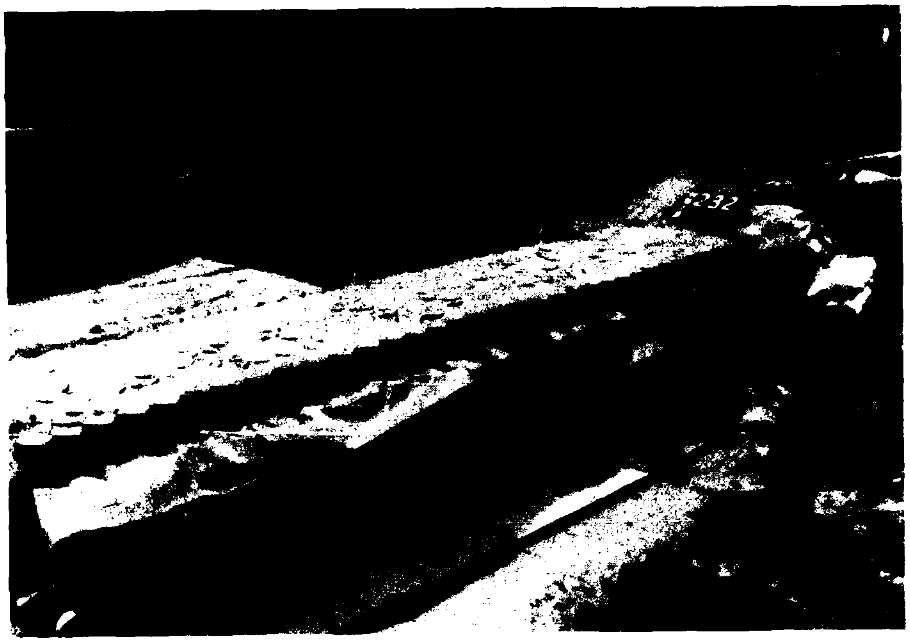

Photo 233. Landside view of Section 6-1 before testing

Photo 234. Trench preparation prior to placing first lift of Section 6-1

Photo 235. First lift in place and being filled and lightly tamped

Photo 236. Filling and screeding of fourth lift

Photo 237. Filling of top lift

Photo 238. Riverside view of Section 6-1 at the end of the static differential head tests

Photo 239. Riverside close-up of cell breakage in top lift

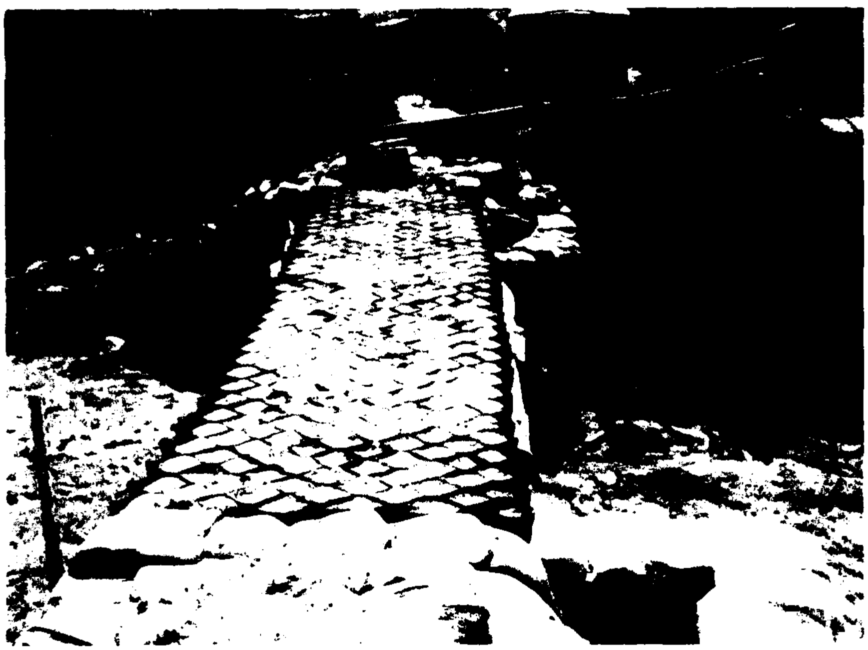

Photo 240. End view of Section 6-1 at the end of the static differential head tests

Photo 241. Landside view of Section 6-1 at the end of the static differential head tests

Photo 242. Closeup of riverside cell breakage on Section 6-1 after 5 hr of wave action at the 3.0-ft water level

Photo 243. Riverside view of cell breakage on Section 6-1 after 18 hr of wave action at the 3.0-ft water level

Photo 244. Sand buildup at riverside toe of Section 6-1 after 18 hr of wave action at the 3.0-ft water level

Photo 245. Riverside view of Section 6-1 after 18 hr of wave action at the 3.0-ft water level

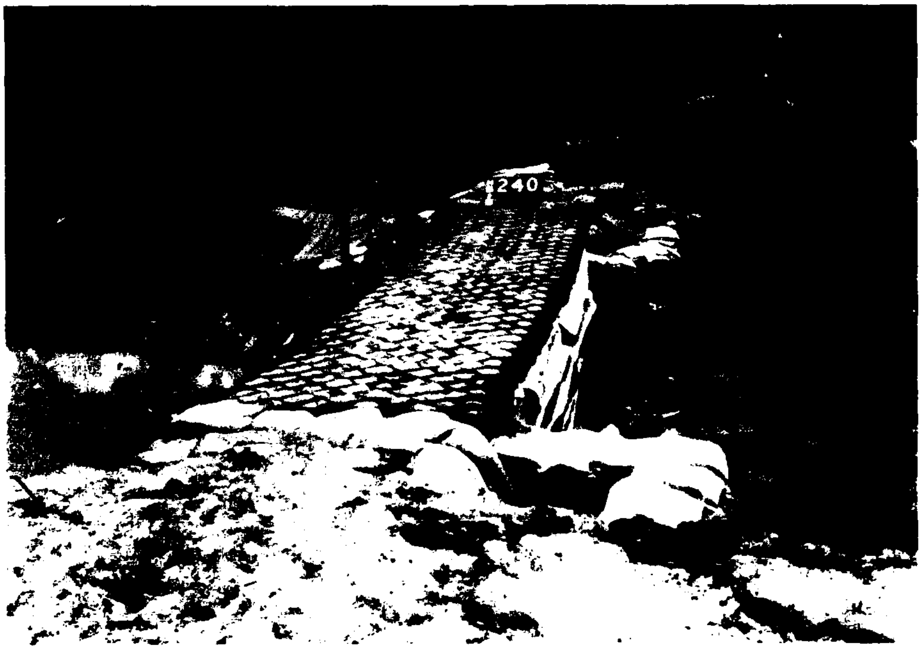

Photo 246. End view of Section 6-1 after 18 hr of wave action at the 3.0-ft water level

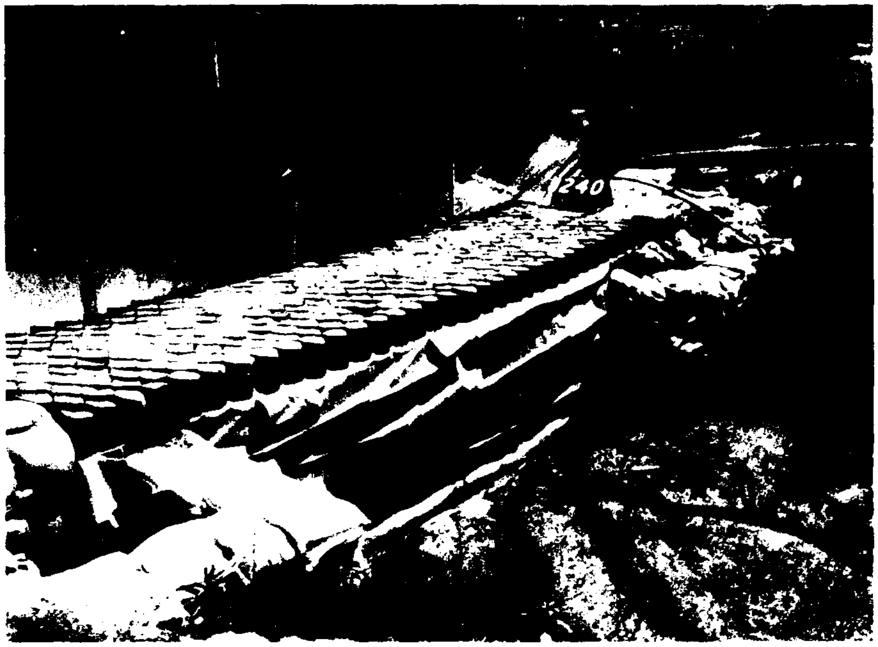

Photo 247. Landside view of Section 6-1 after 18 hr of wave action at the 3.0-ft water level

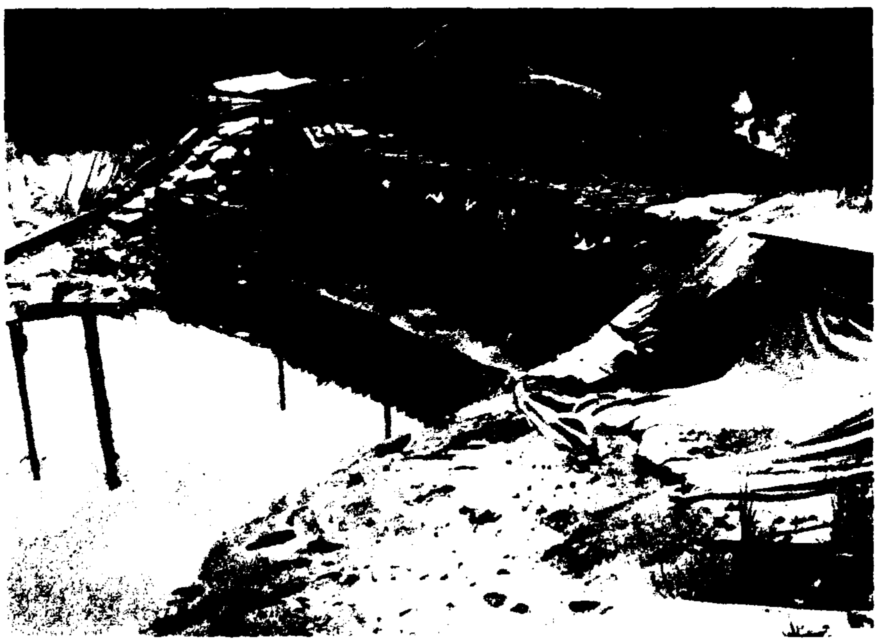

Photo 248. Riverside view of Section 6-1 after 19 hr of wave action at the 4.0-ft water level

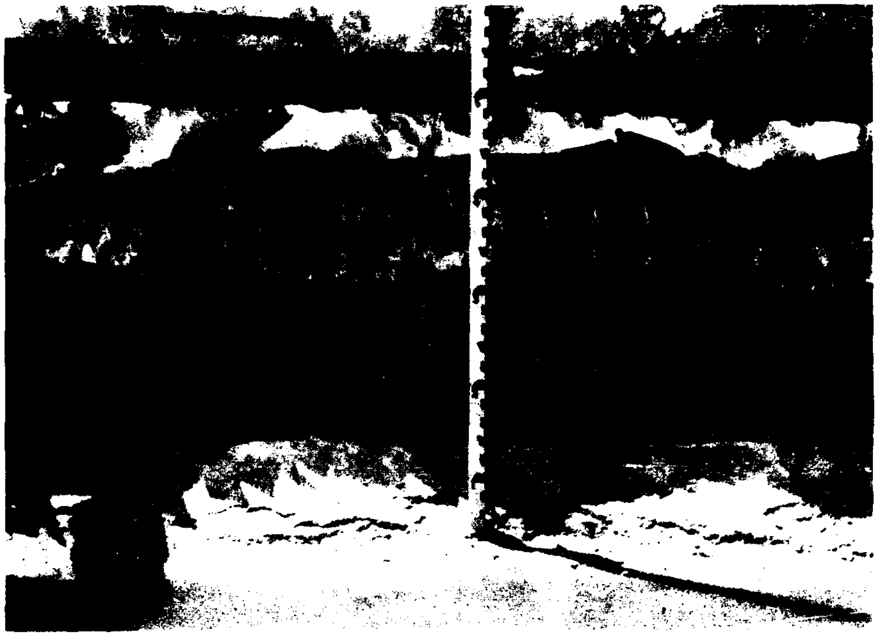

Photo 249. Riverside cell breakage and sand buildup at toe of Section 6-1 after 19 hr of wave action at the 4.0-ft water level

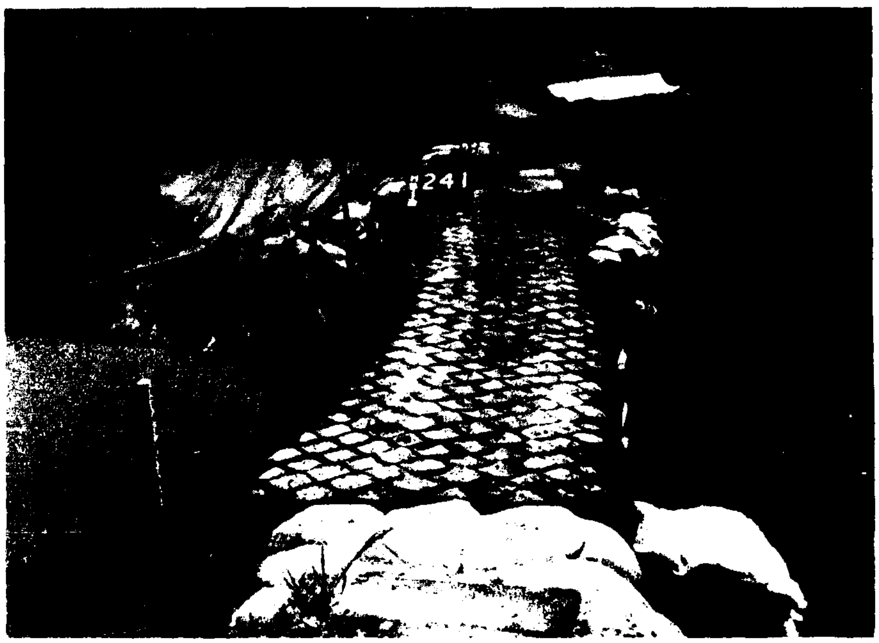

Photo 250. End view of Section 6-1 after 19 hr of wave action at the 4.0-ft water level (Note bow in structure.)

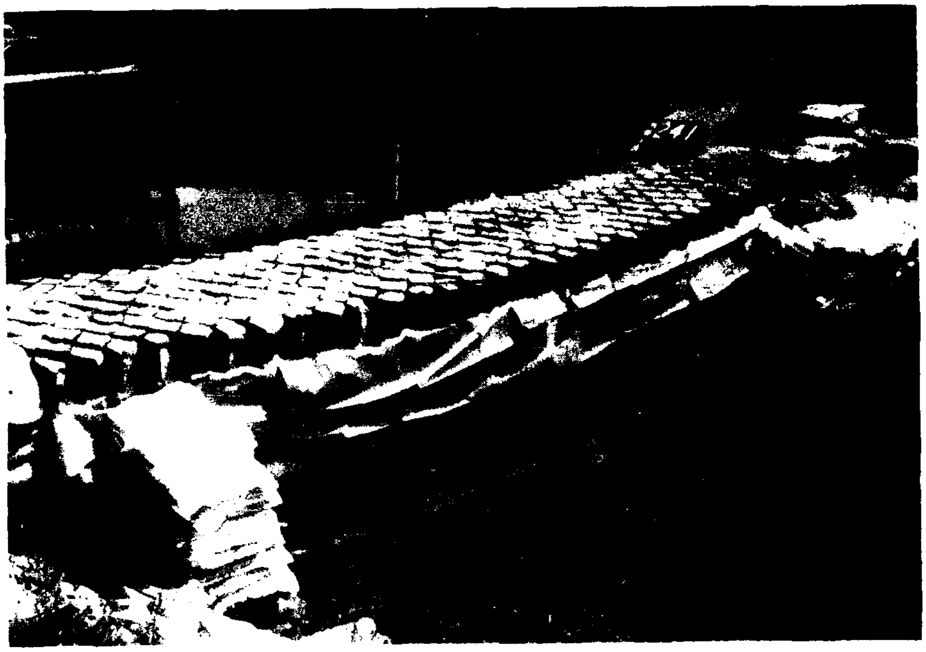

Photo 251. Landside view of Section 6-1 after 19 hr of wave action at the 4.0-ft water level

Photo 252. During failure of Section 6-1

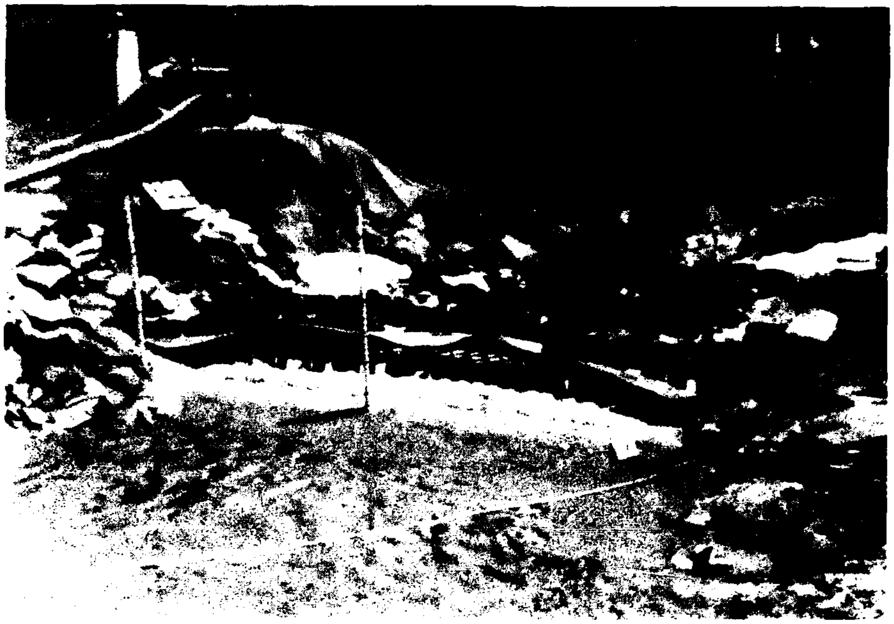

Photo 253. Riverside view of Section 6-1 at end of test

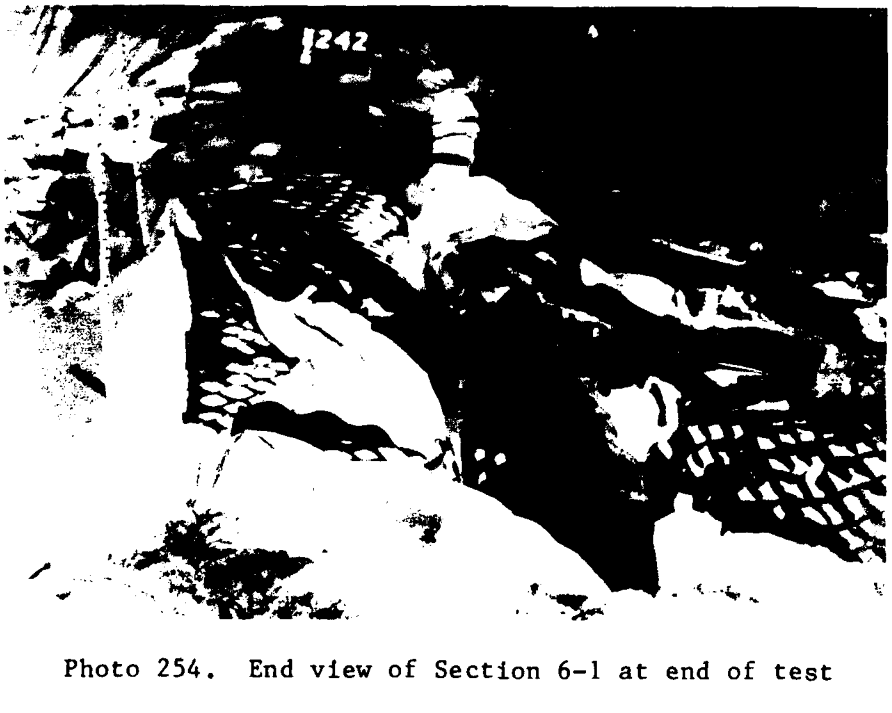

Photo 254. End view of Section 6-1 at end of test

Photo 255. Riverside view of Section 6-2 before testing

Photo 256. End view of Section 6-2 before testing

Photo 257. Landside view of Section 6-2
before testing

Photo 258. Placement of support posts on Section 6-2

Photo 259. Nailing planking on Section 6-2

Photo 260. Filling of Section 6-2

Photo 261. Distributing and tamping earth fill (clayey silt) on Section 6-2

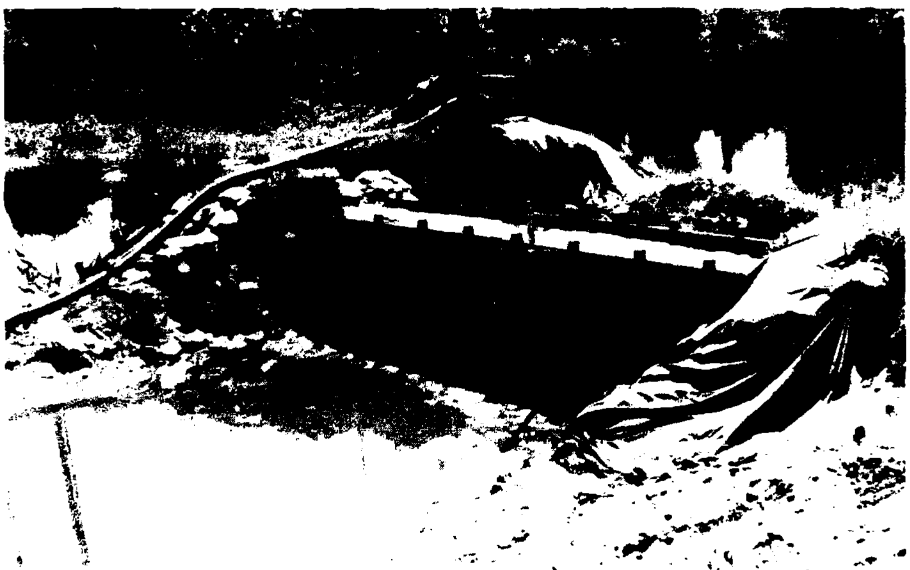

Photo 262. Riverside view of Section 6-2 at the end of the static differential head tests

Photo 263. End view of Section 6-2 at the end of the static differential head tests

Photo 264. Landside view of Section 6-2 at the end of the static differential head tests

Photo 265. End view of wave overtopping on Section 6-2 at the 4.0-ft water level

Photo 266. Landside view of water flowing around center support posts during wave action at the 4.0-ft water level

Photo 267. End view of erosion sustained by earth fill during 8.5 hr of wave action at the 4.0-ft water level (end of test)

Photo 268. Riverside view of Section 6-2 after 8.5 hr of wave action at the 4.0-ft waver level (end of test)

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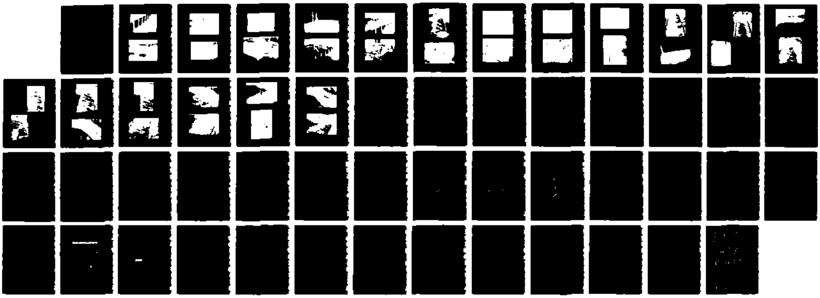

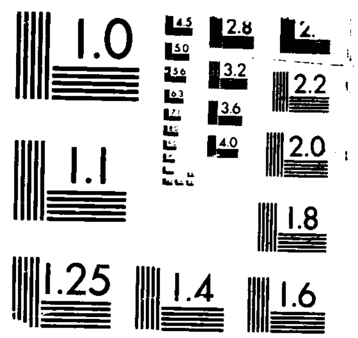

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Photo 269. Landside view of Section 6-2 after 8.5 hr of wave action at the 4.0-ft waver level (end of test)

Photo 270. Riverside view of Section 6-3 before testing

Photo 271. Landside view of Section 6-3 before testing

Photo 272. Riverside view of Section 6-3 during the 1.0-ft static differential head test

Photo 273. Landside view of Section 6-3 during the 1.0-ft static differential head test

Photo 274. Riverside view of Section 6-3 after failure at the 2.0-ft static differential head

Photo 275. Close-up of point where water undermined the riverside of Section 6-3 during the 2.0-ft static differential head

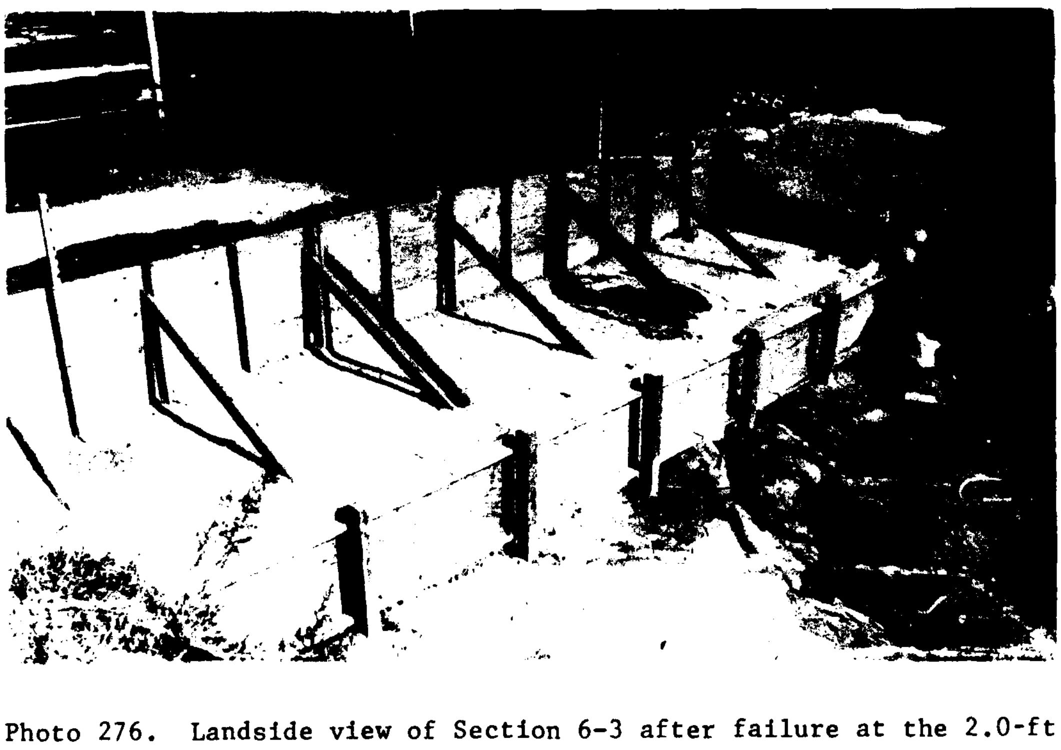


Photo 276. Landside view of Section 6-3 after failure at the 2.0-ft static differential head

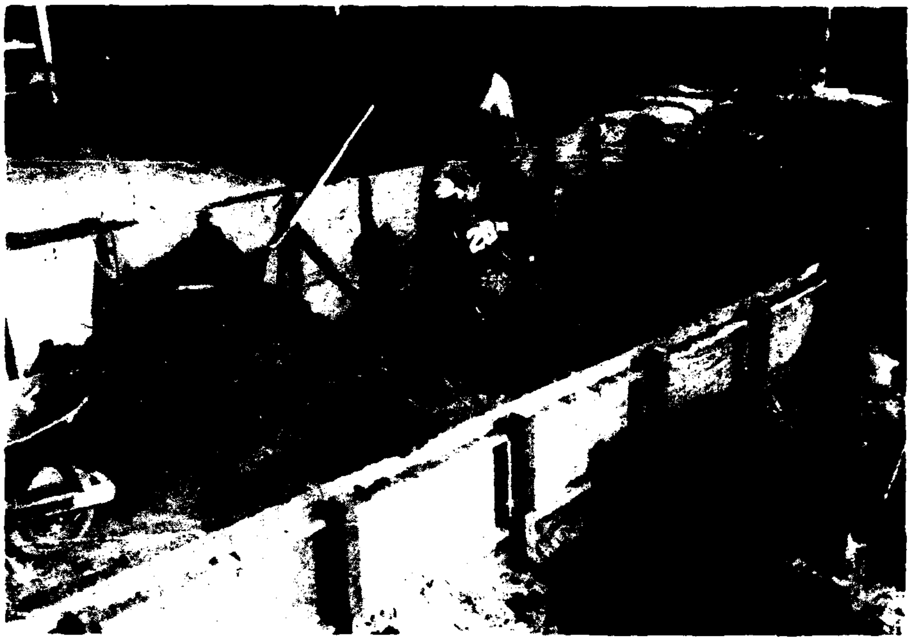


Photo 277. Landside view during the preparation of Section 6-3-A

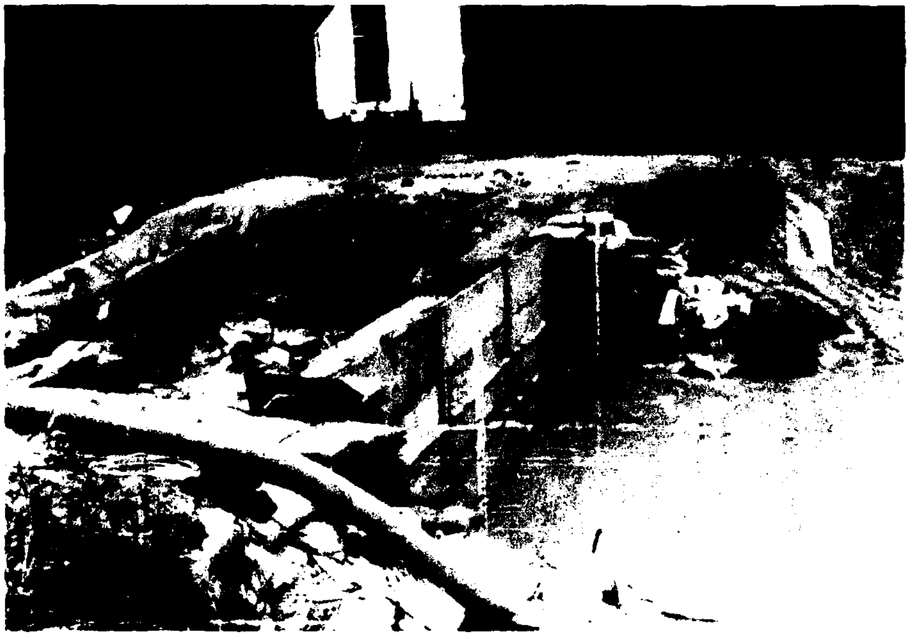


Photo 278. Riverside view of Section 6-3-A before testing

Photo 279. End view of Section 6-3-A before testing

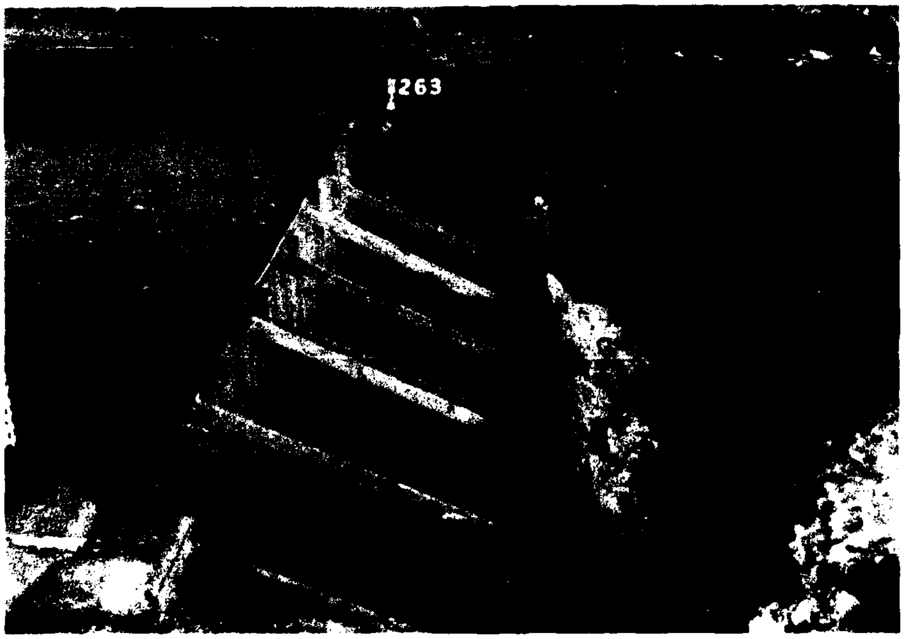


Photo 280. End view of Section 6-3-A during its failure
at the 5.0-ft static differential head

Photo 281. Riverside view of Section 6-4 before testing

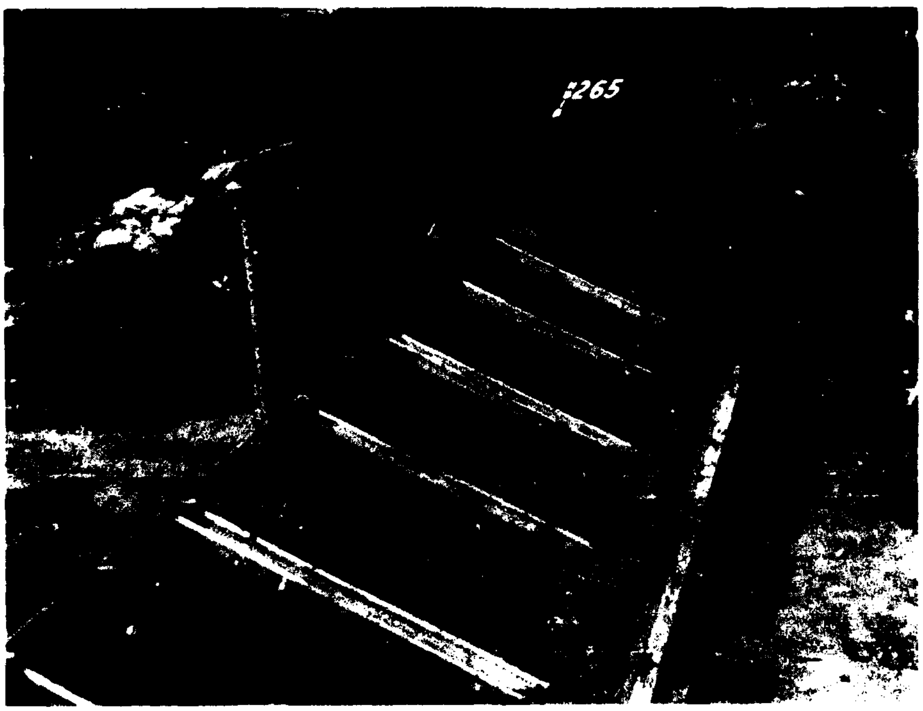


Photo 282. End view of Section 6-4 before testing

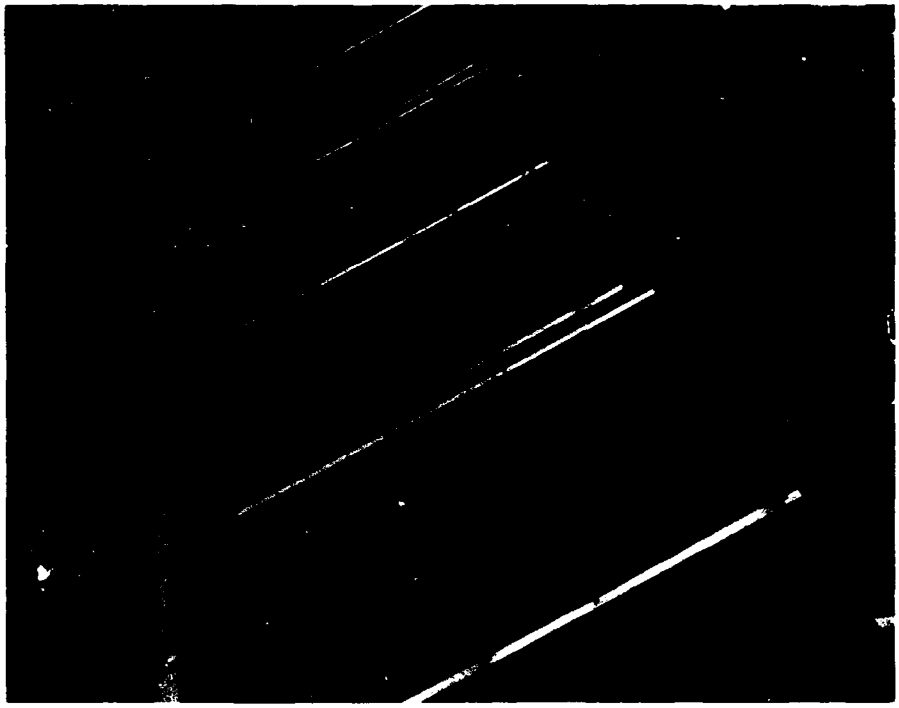


Photo 283. End view of Section 6-4 in-place and prior to filling trench and placing earth fill

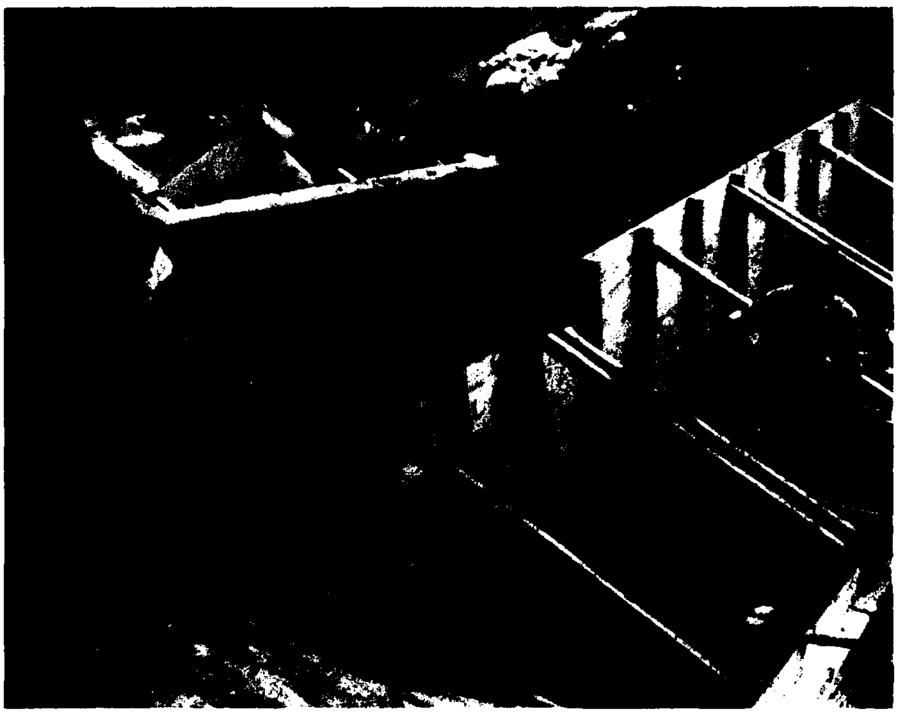


Photo 284. End view of Section 6-4 during placement of earth fill

Photo 285. Landside view of Section 6-4 at the end of the 1.0-ft static differential head test

Photo 286. Landside view of Section 6-4 at the end of the 2.0-ft static differential head test

Photo 287. End view of Section 6-4 after failure during the 3.0-ft static differential head test (end of test)

Photo 288. Close-up of eroded trench fill on riverside of Section 6-4

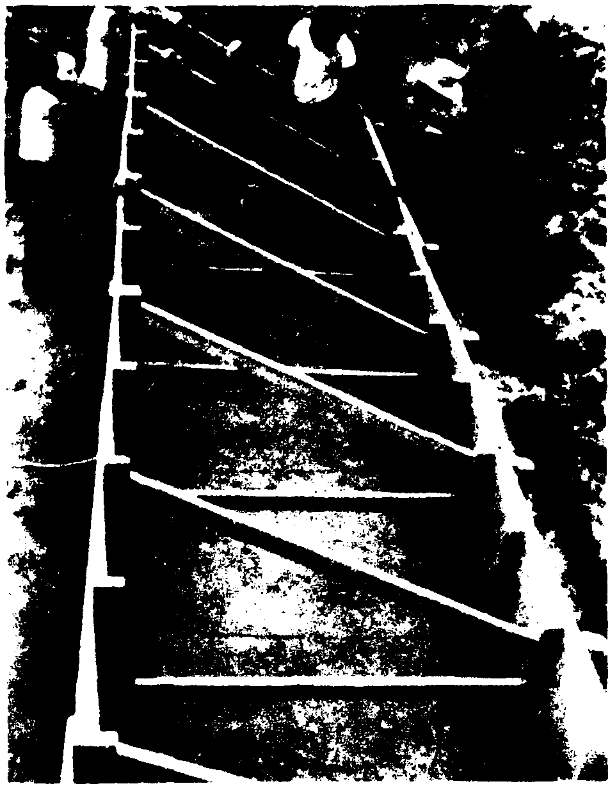


Photo 289. End view of Section 6-5 in-place and prior to filling trench and placing earth fill

Photo 290. End view of Section 6-5 during placement of earth fill

Photo 291. Riverside view of Section 6-5 before testing

Photo 292. End view of Section 6-5 before testing

Photo 293. End view of Section 6-5 at the end of the 3.0-ft static differential head test

Photo 294. End view of Section 6-5 after 2.0 hr of the 4.0-ft static differential head test

Photo 295. End view of Section 6-5 at the end
of the 4.0-ft static differential head test

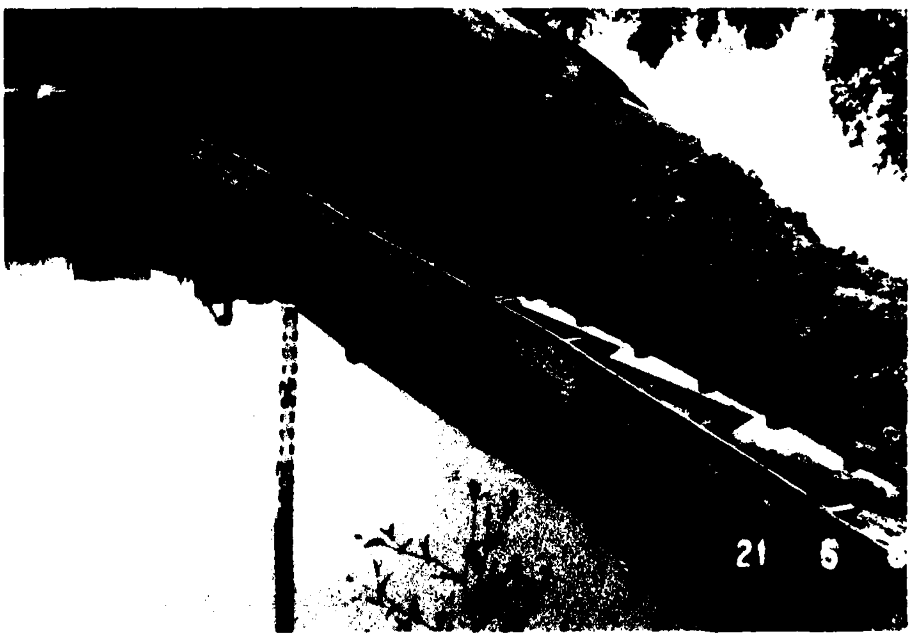


Photo 296. Riverside view of Section 6-5 at the end of
the 5.0-ft static differential head test

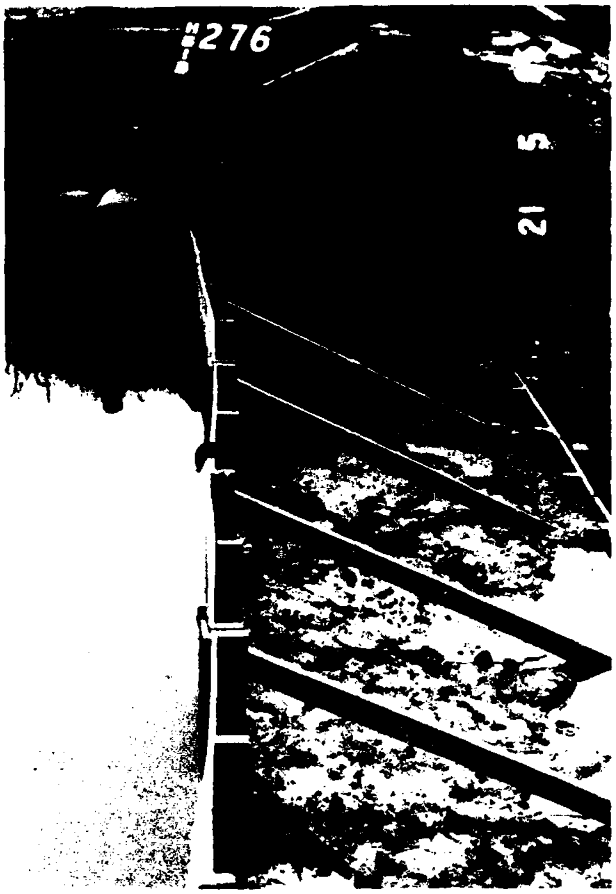


Photo 297. End view of Section 6-5 at the end of the 5.0-ft static differential head test

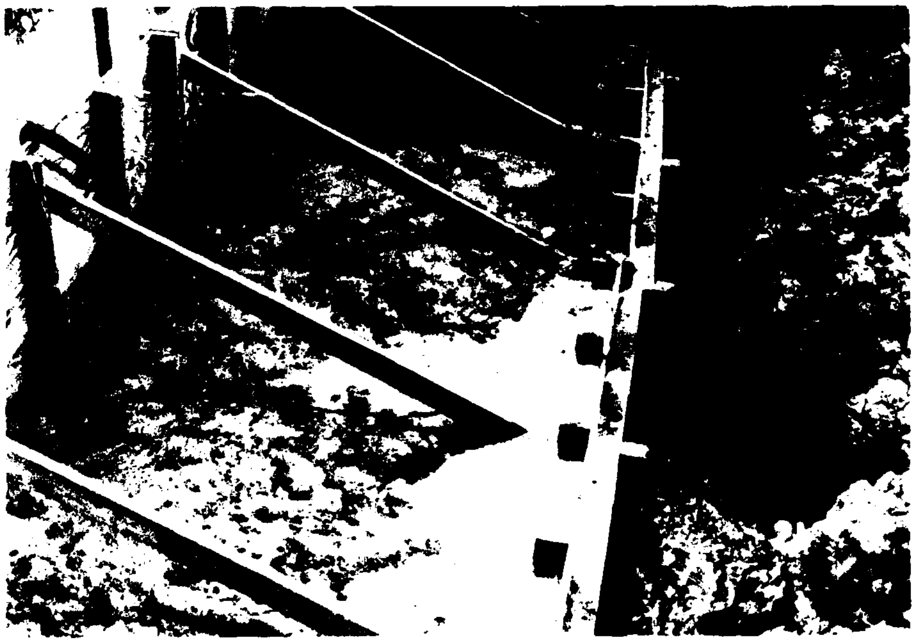


Photo 298. Landside close-up of Section 6-5 at the end of the 5.0-ft static differential head test

Photo 299. Riverside view of Section 6-5 during wave attack at the 4.0-ft water level

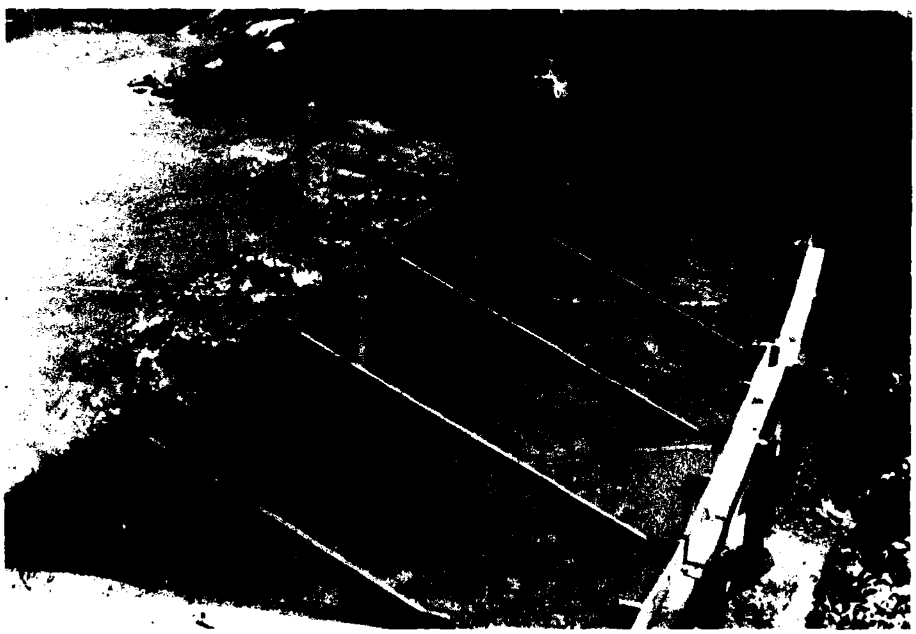


Photo 300. End view of Section 6-5 during wave attack at the 4.0-ft water level

Photo 301. End view showing Section 6-5 holding a 5.5-ft static differential head

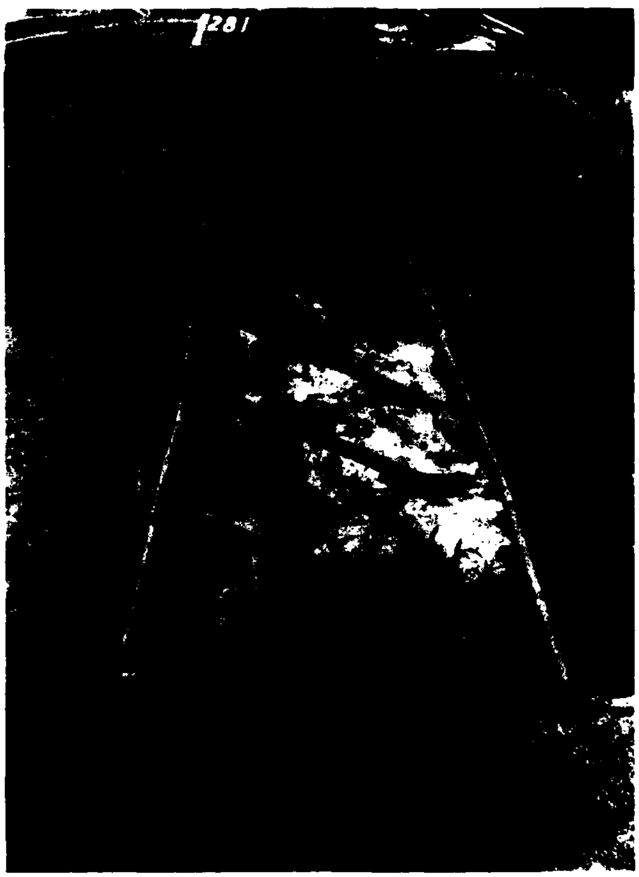


Photo 302. Riverside view of Section 6-5 at end of test

Photo 303. End view of Section 6-5 at end of test

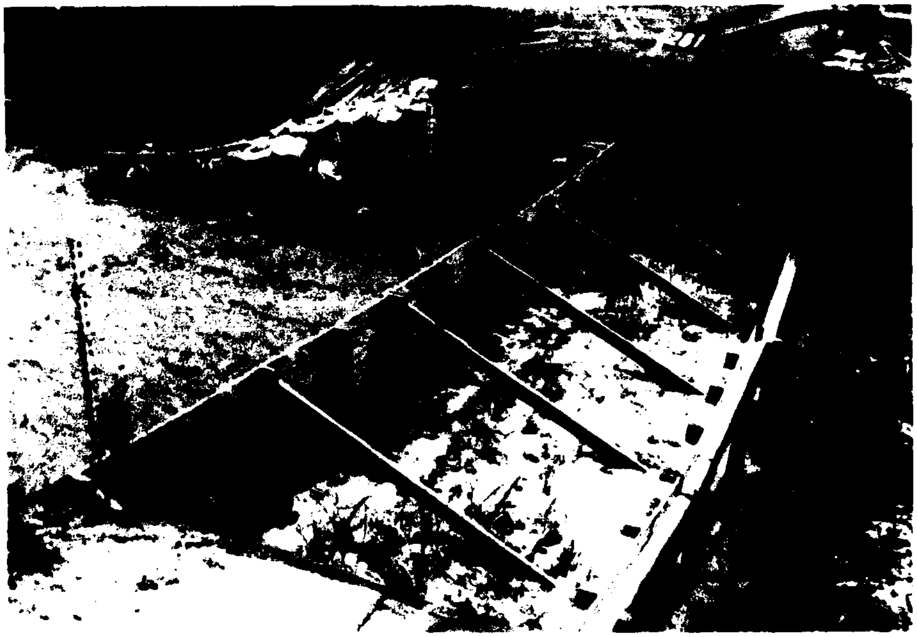
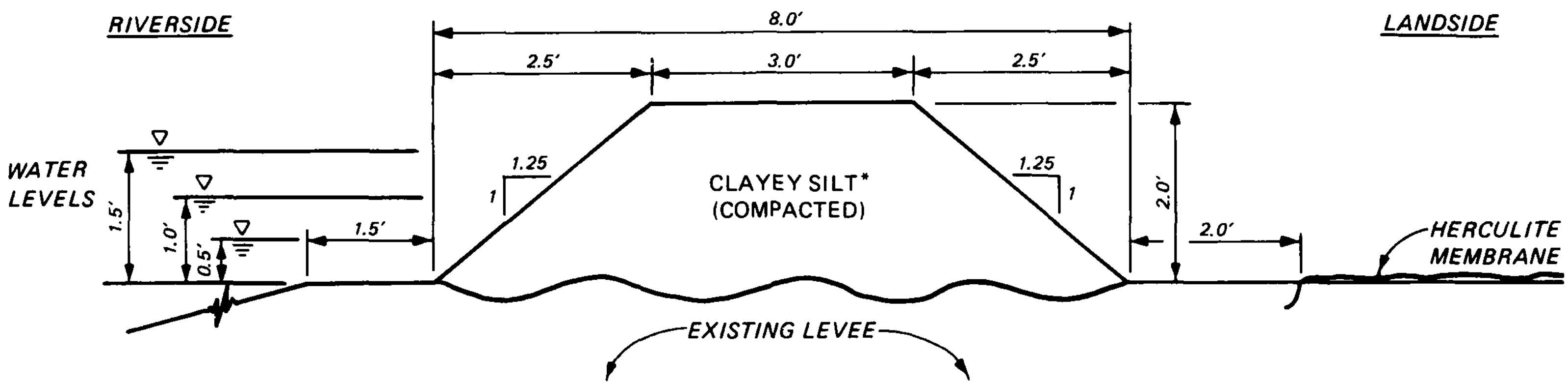
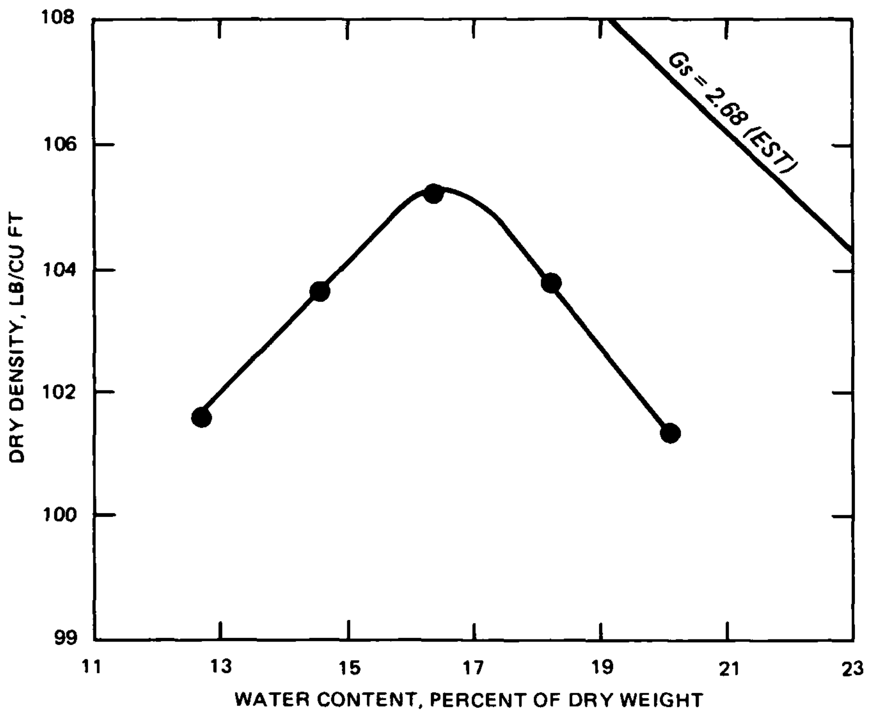


Photo 304. Landside view of Section 6-5 at end of test



* WET DENSITY (AVERAGE) = 110.1 PCF
DRY DENSITY (AVERAGE) = 91.2 PCF
WATER CONTENT (AVERAGE), PERCENT OF DRY WEIGHT = 20.8%

SECTION 2-1
2-FT-HIGH POTATO RIDGE



STANDARD COMPACTION TEST

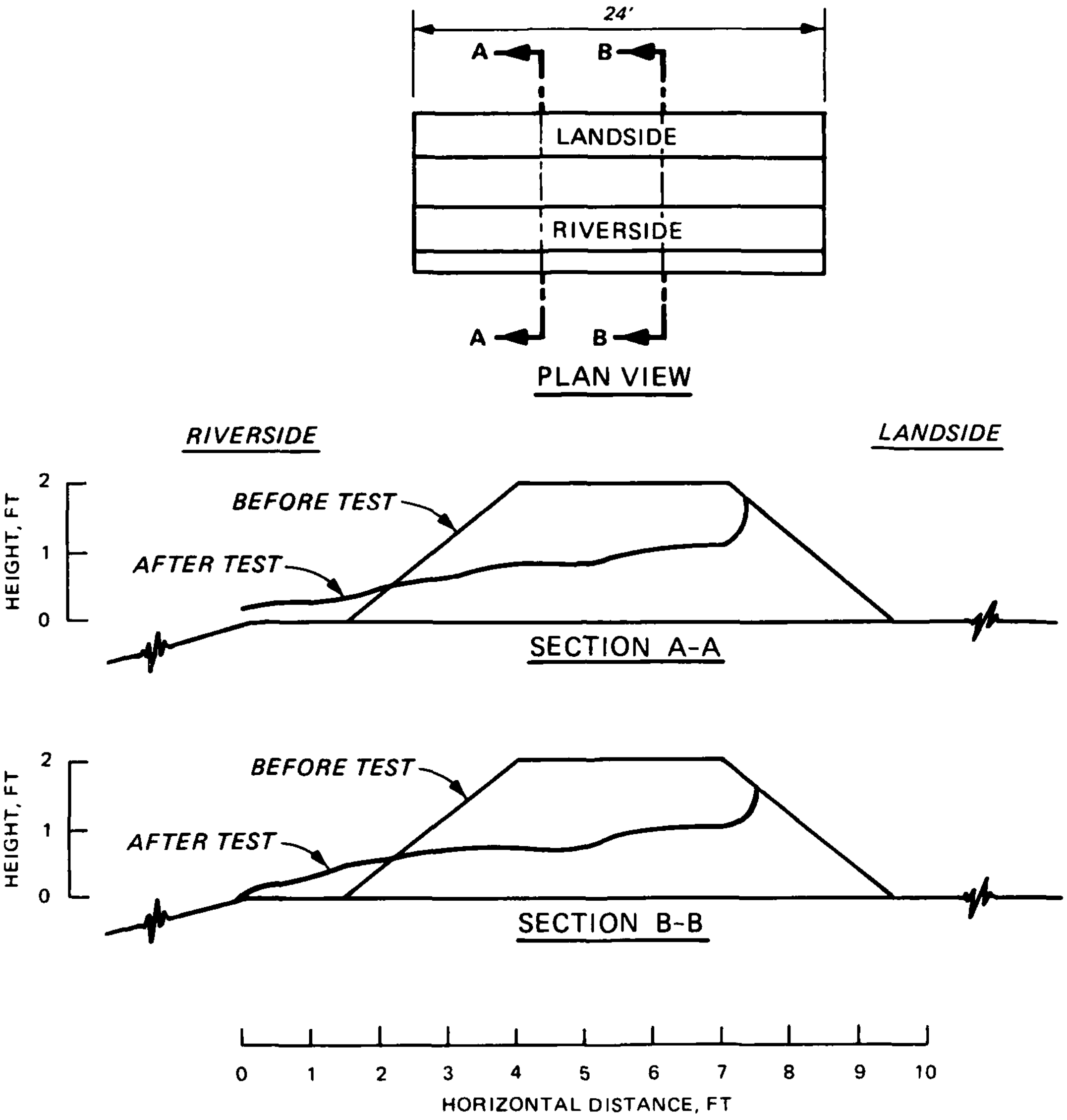
15 BLOWS PER EACH OF 3 LAYERS, WITH 5.5 LB SLIDING WT

12 INCH DROP. 4 INCH DIAMETER MOLD

SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	G	LL	PL	% > NO. 4	% > 3/4 IN.
		CLAYEY SILT (ML), BROWN	2.68				
			(EST)				

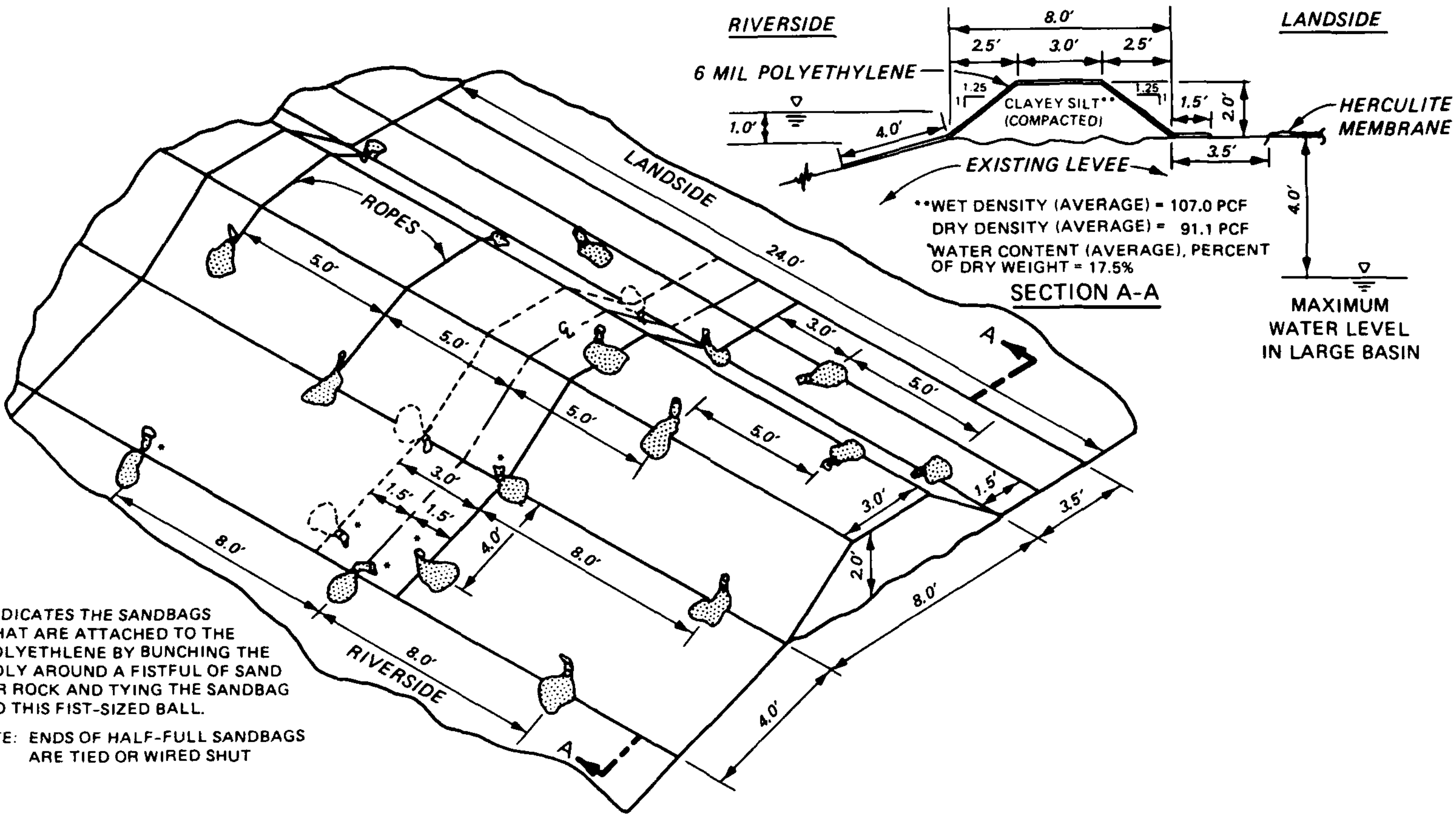
CLAYEY SILT
COMPACTION TEST
AND
VISUAL CLASSIFICATION

PLATE 2



*AFTER 19 HR OF WAVE ACTION AT THE 1.0 FT-WATER LEVEL

BEFORE AND AFTER TEST*
CROSS SECTIONS OF SECTION 2-1

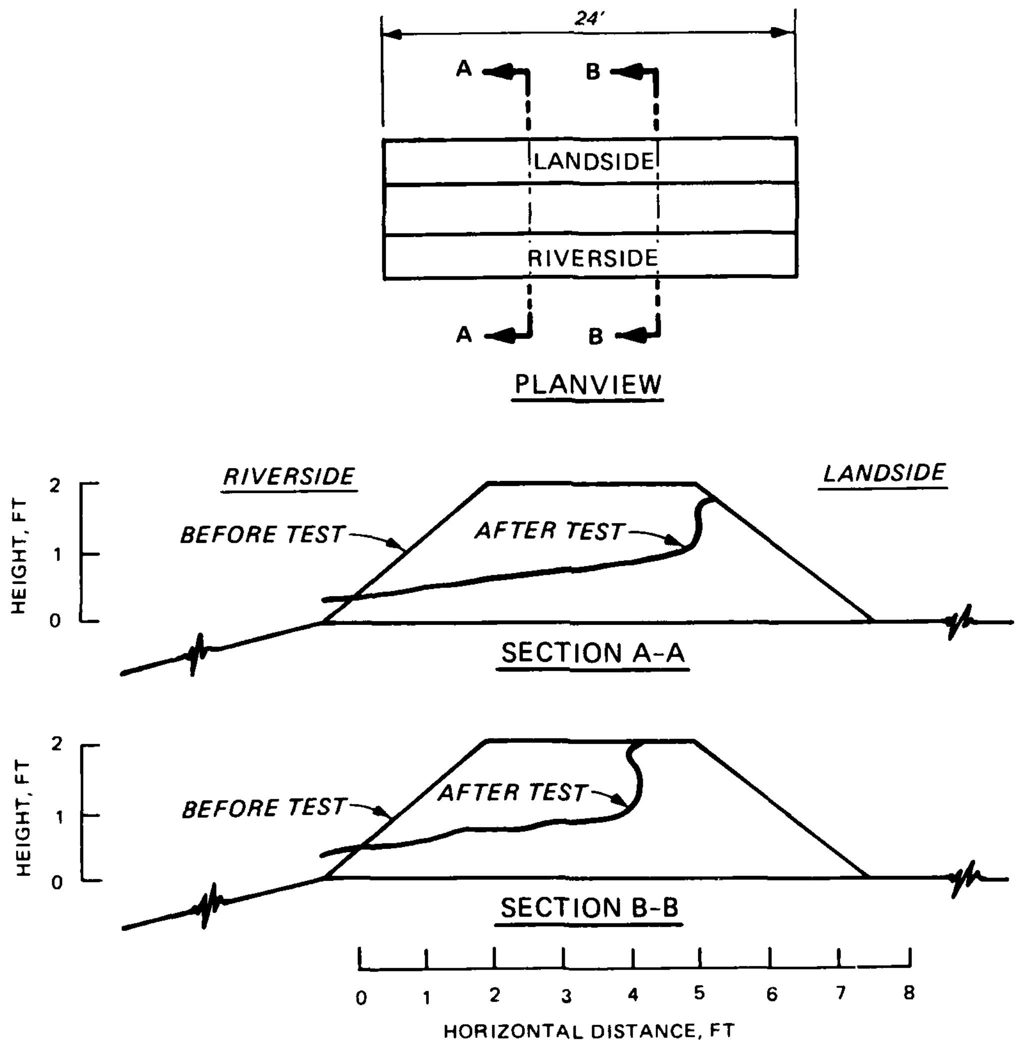


* INDICATES THE SANDBAGS THAT ARE ATTACHED TO THE POLYETHLENE BY BUNCHING THE POLY AROUND A FISTFUL OF SAND OR ROCK AND TYING THE SANDBAG TO THIS FIST-SIZED BALL.

NOTE: ENDS OF HALF-FULL SANDBAGS ARE TIED OR WIRED SHUT

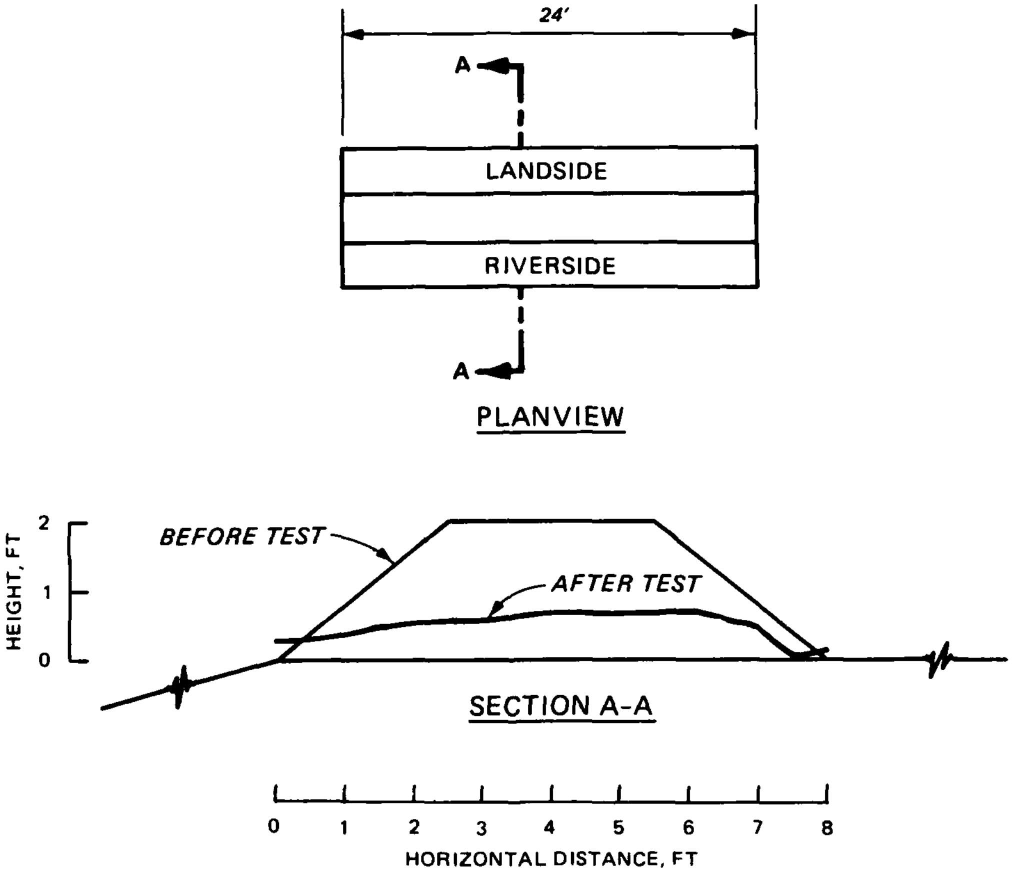
SECTION 2-2
2-FT-HIGH POTATO RIDGE
WITH
POLYETHYLENE COVERING

PLATE 4



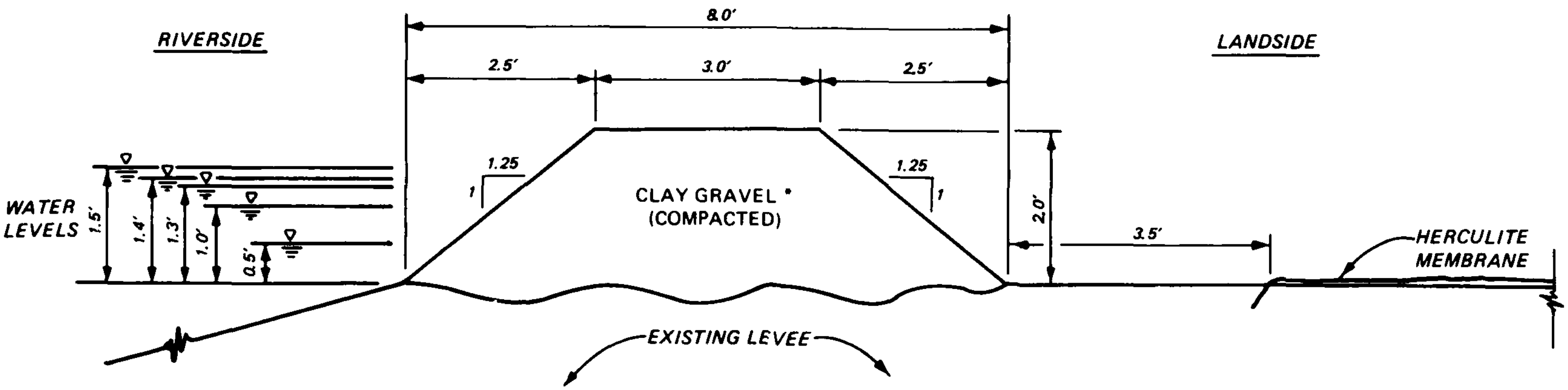
*AFTER 19 HR OF WAVE ACTION AT THE 1.0 FT WATER LEVEL

BEFORE AND AFTER TEST*
CROSS SECTIONS OF SECTION 2-2



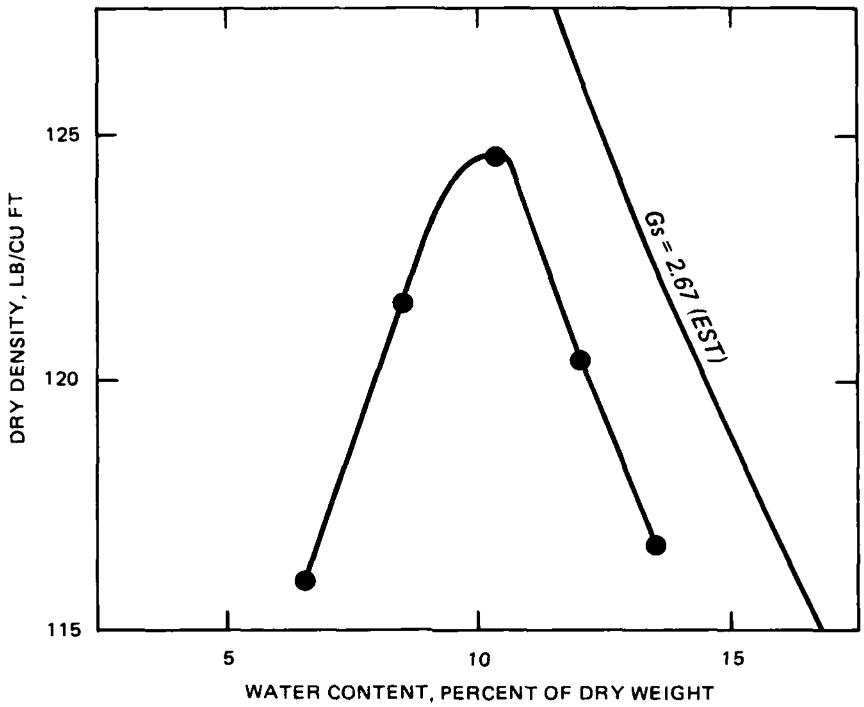
*AFTER 34-1/2 HR OF WAVE ATTACK AT THE
1.0 FT WATER LEVEL

BEFORE AND AFTER TEST*
CROSS SECTIONS OF SECTION 2-2
(END OF TEST)



* WET DENSITY (AVERAGE) = 122.0 PCF
DRY DENSITY (AVERAGE) = 112.3 PCF
WATER CONTENT (AVERAGE), PERCENT OF DRY WEIGHT = 8.6 %

SECTION 2-3
2-FT-HIGH POTATO RIDGE



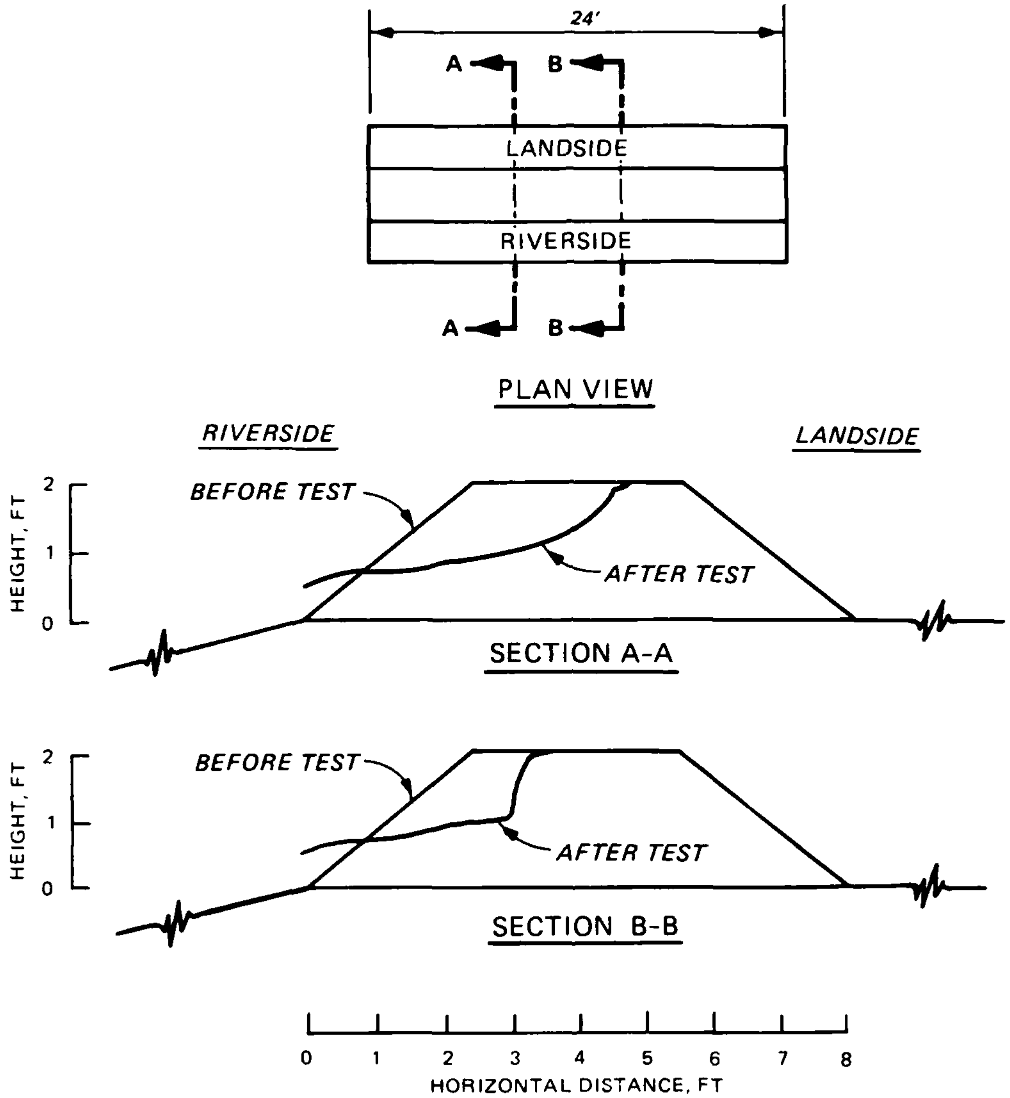
STANDARD COMPACTION TEST

56 BLOWS PER EACH OF 3 LAYERS, WITH 5.5 LB. SLIDING WT.

12 INCH DROP, 6 INCH DIAMETER MOLD

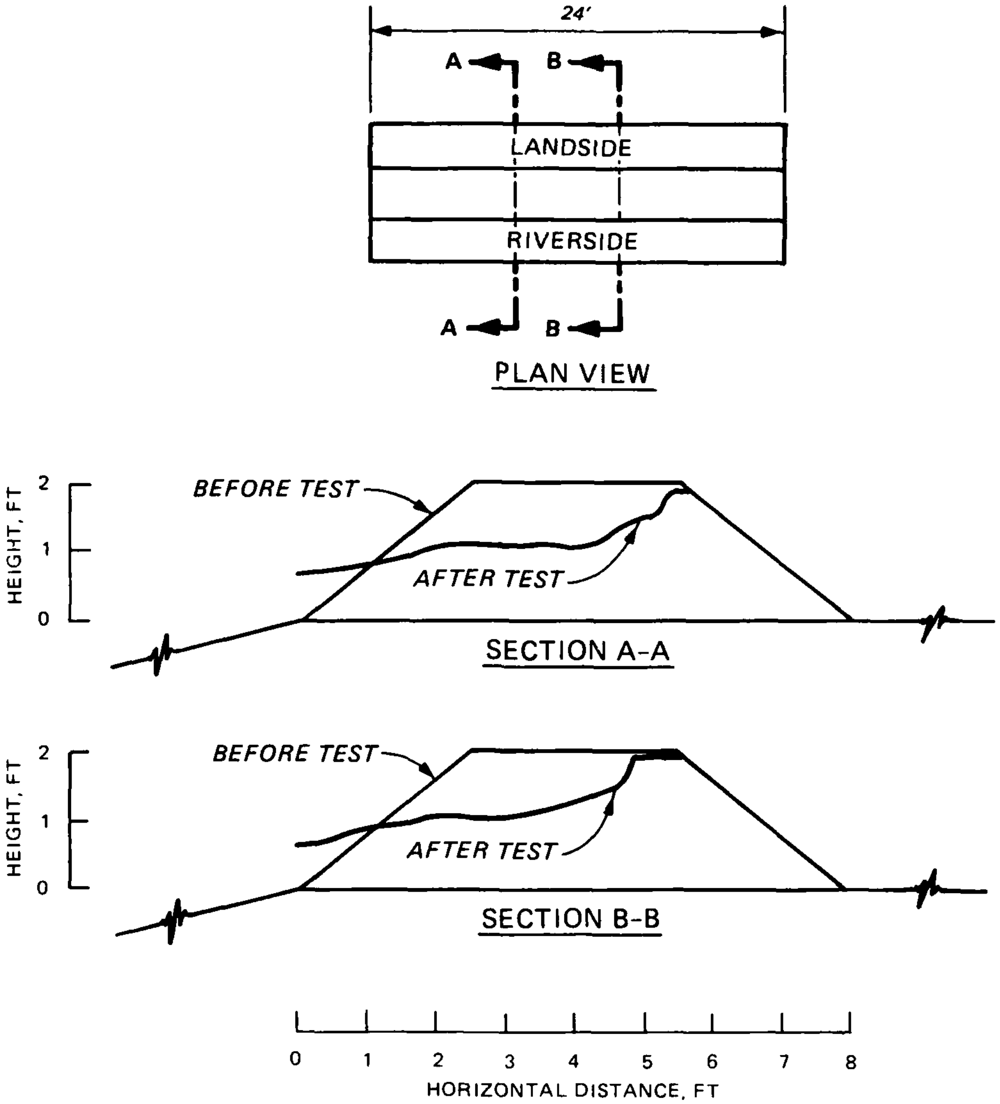
SAMPLE NO.	ELEV OR DEPTH	CLASSIFICATION	G	LL	PL	% > NO. 4	% > 3/4 IN.
		GRAVELLY CLAYEY SAND (SC)	2.67				
			(EST)				

CLAY GRAVEL COMPACTION TEST AND VISUAL CLASSIFICATION



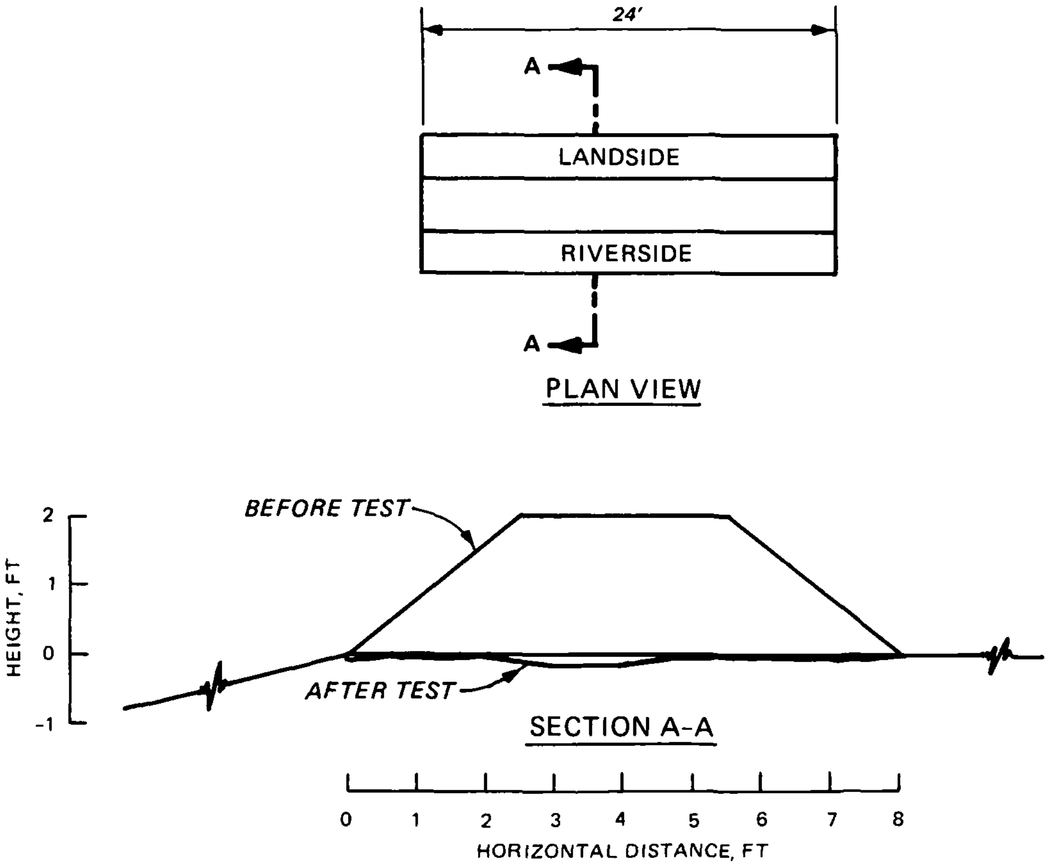
*AFTER 19 HR OF WAVE ACTION AT THE 1.0 FT WATER LEVEL

SECTION 2-3
BEFORE AND AFTER TEST*
CROSS SECTIONS
(1.0-FT WATER LEVEL)



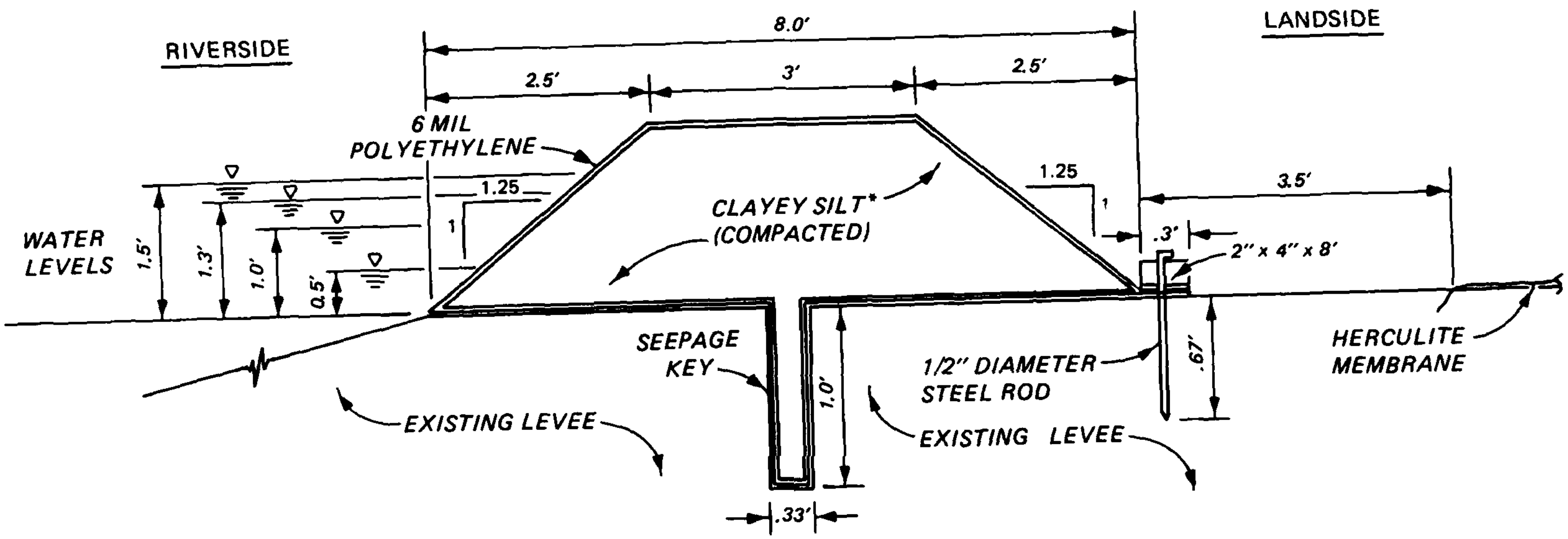
*AFTER 12-1/2 HR OF WAVE ACTION AT THE 1.3 FT WATER LEVEL

SECTION 2-3
BEFORE AND AFTER TEST*
CROSS SECTIONS
(1.3-FT WATER LEVEL)



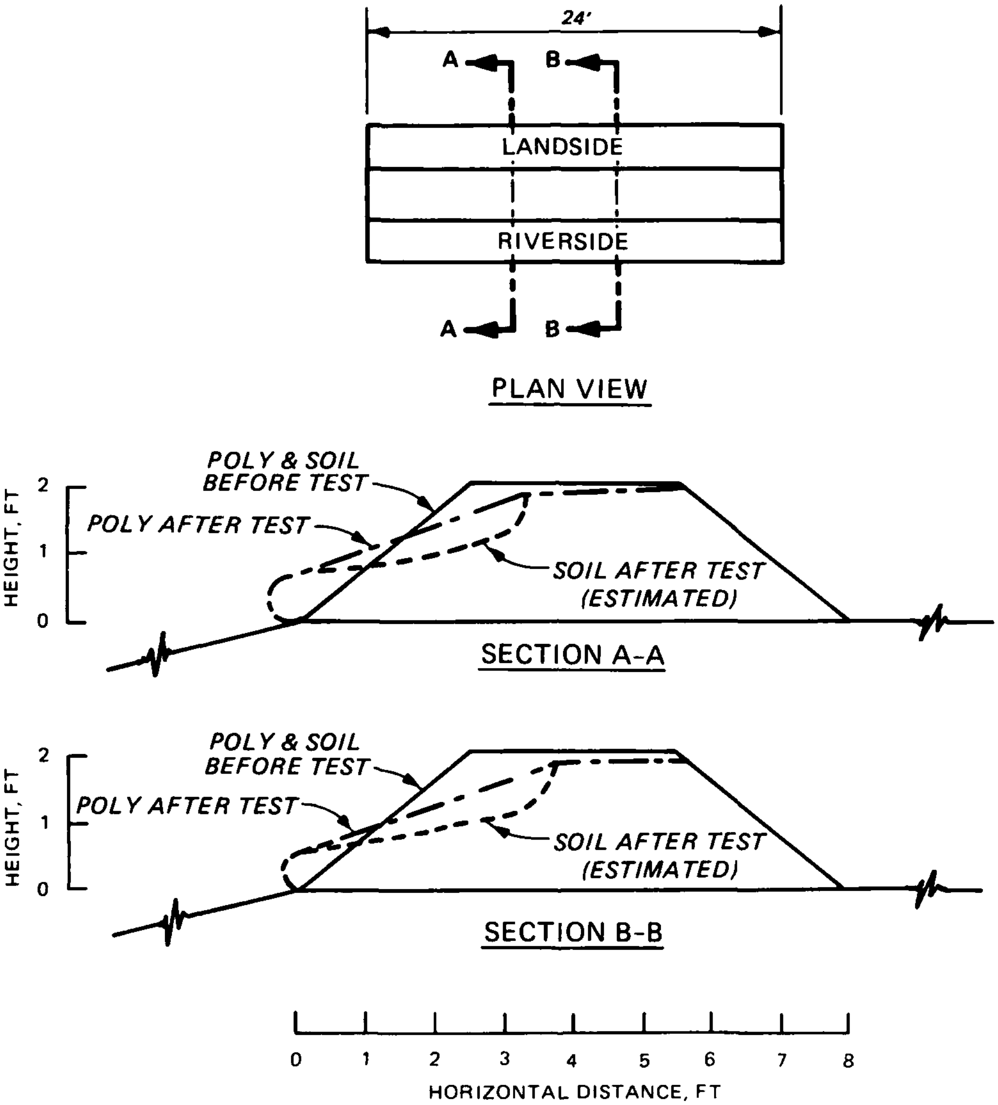
*AFTER 0.62 HR OF WAVE ACTION AT THE 1.4 FT WATER LEVEL

SECTION 2-3
BEFORE AND AFTER TEST *
CROSS SECTIONS
(1.4-FT WATER LEVEL)



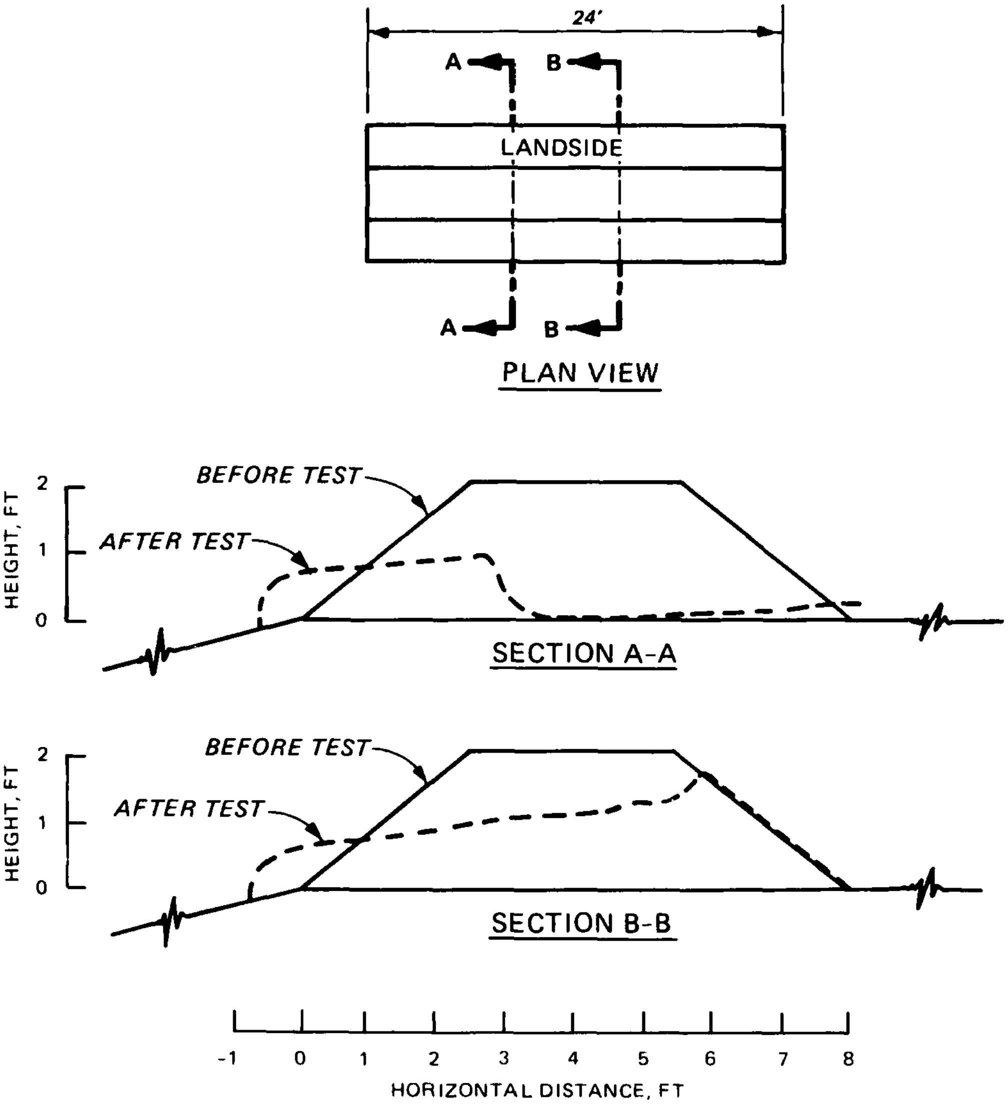
* WET DENSITY (AVERAGE) = 100.5 PCF
DRY DENSITY (AVERAGE) = 85.7 PCF
WATER CONTENT (AVERAGE), PERCENT OF DRY WEIGHT = 17.2%

SECTION 2-4
2-FT-HIGH POTATO RIDGE ENCASED IN 6 MIL POLYETHYLENE



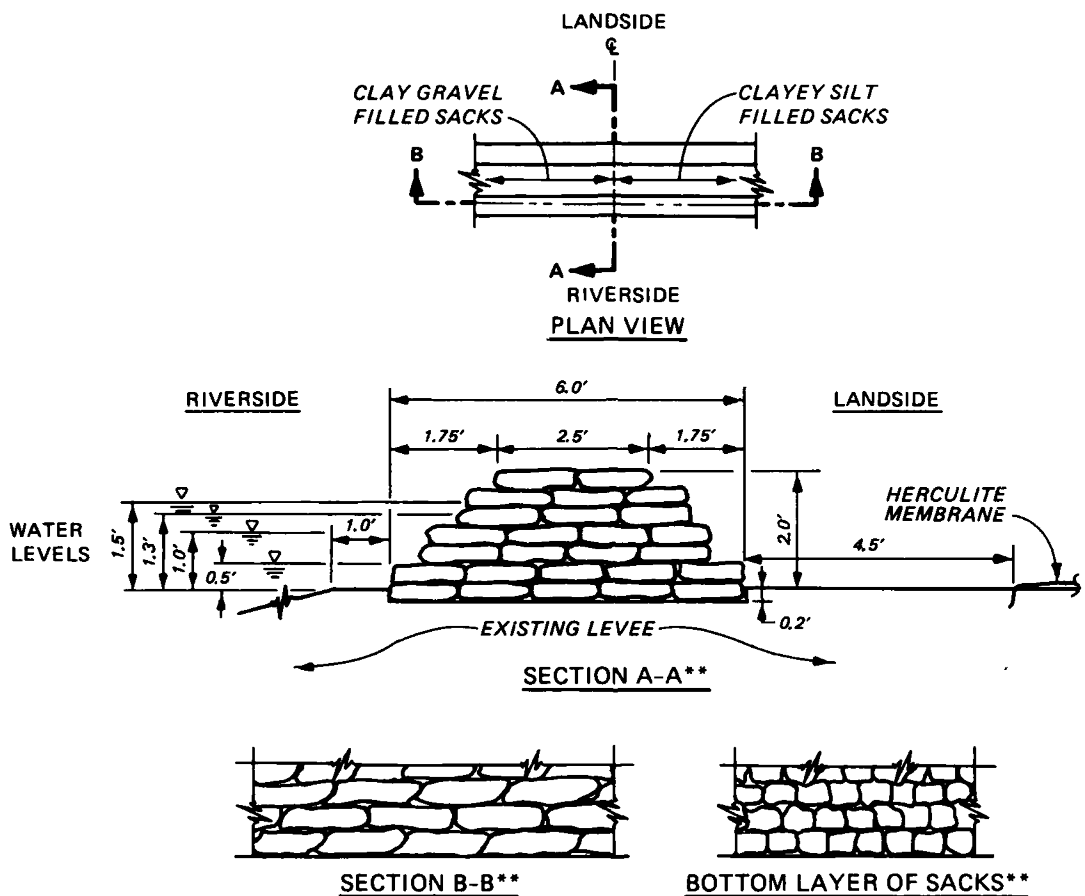
*AFTER 19 HR OF WAVE ACTION AT THE 1.0 FT WATER LEVEL

SECTION 2-4
BEFORE AND AFTER TEST *
CROSS SECTIONS
(1.0-FT WATER LEVEL)



*AFTER 25-1/2 HR OF WAVE ACTION AT THE 1.3 FT WATER LEVEL

SECTION 2-4
BEFORE AND AFTER TEST*
CROSS SECTIONS
(1.3-FT WATER LEVEL)

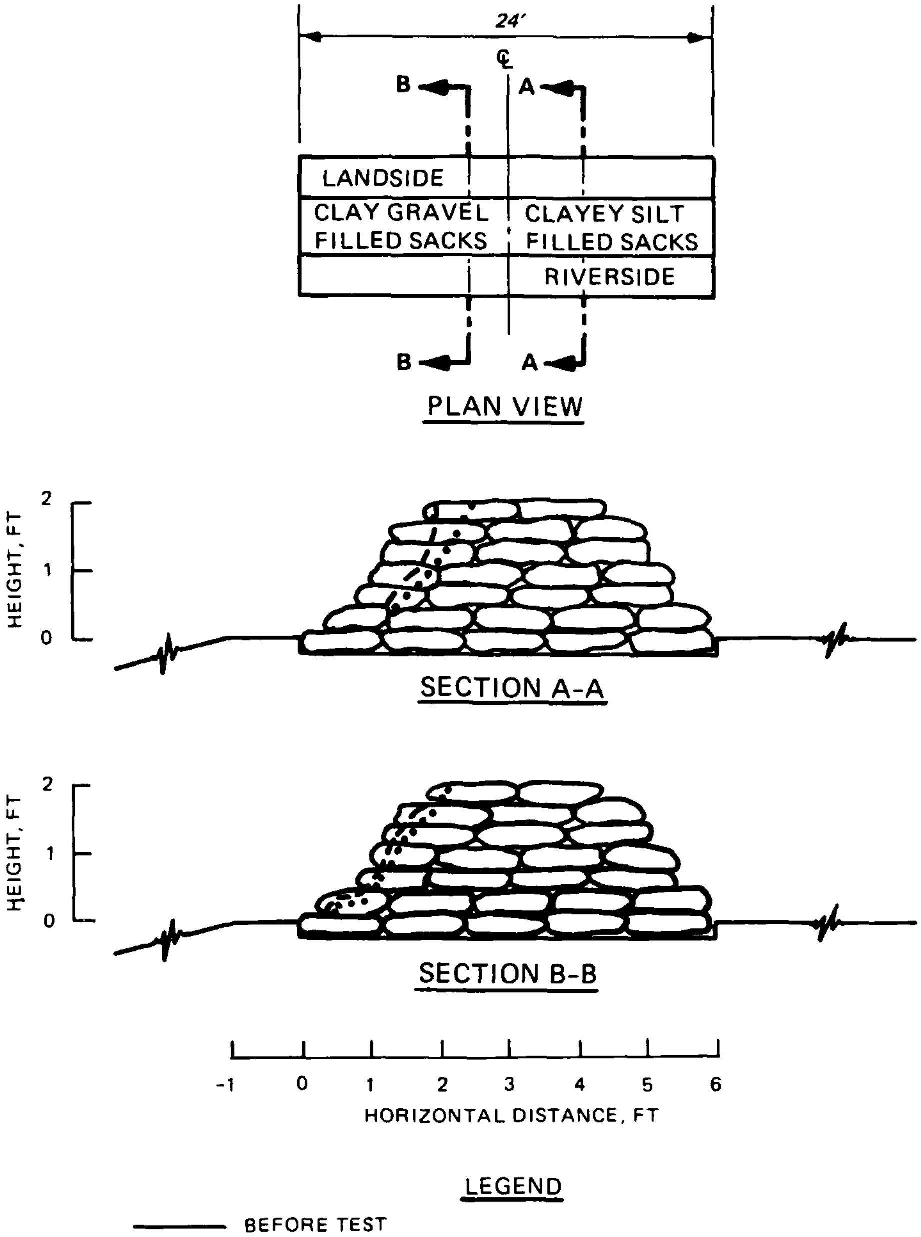


*SACKS ARE ONE-HALF TO TWO-THIRDS FULL AND ARE UNTIED. UNTIED SACK ENDS ARE FOLDED BACK AND THE ADJACENT SACKS CLOSED ENDS ARE LAYED OVER THE FOLDS

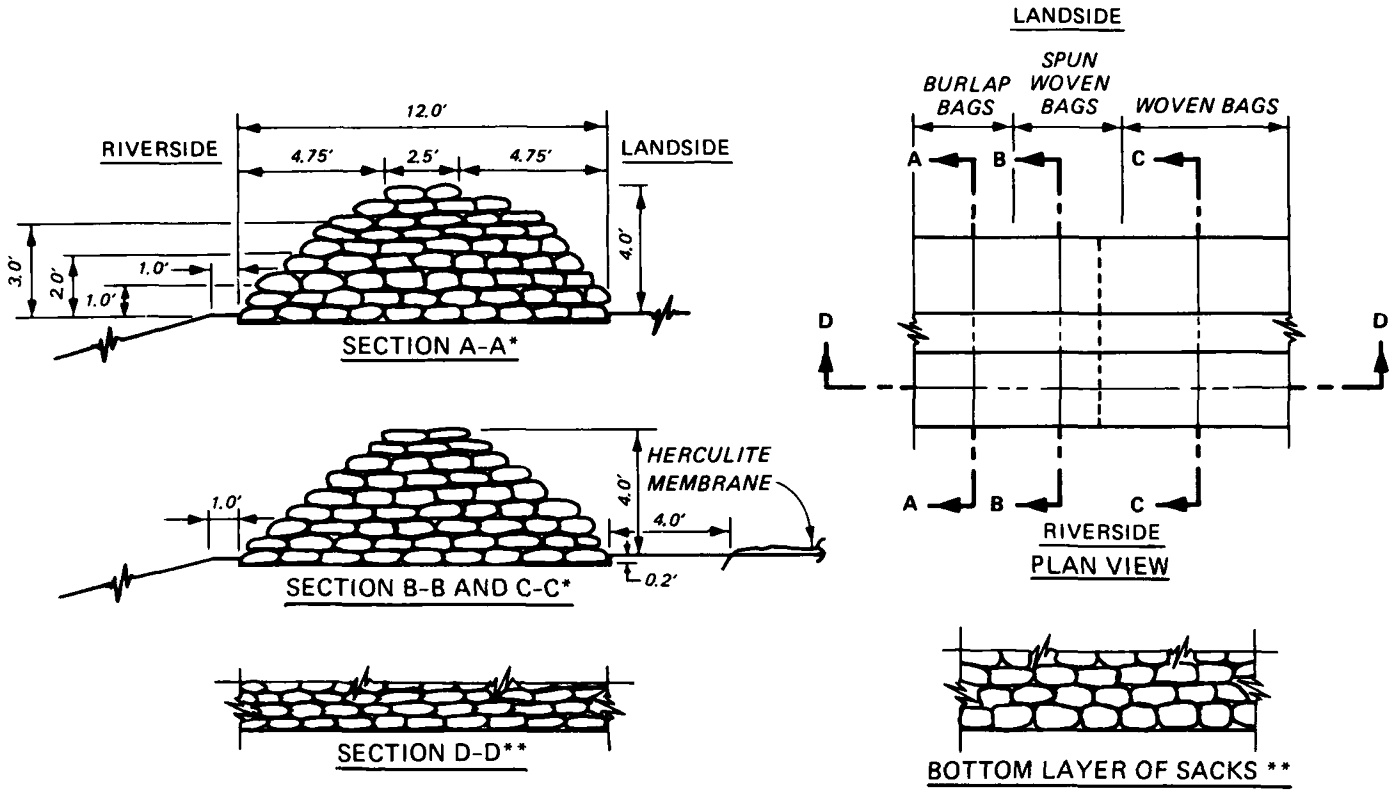
**NOTE OFFSET OF SEAMS TO BREAK SEEPAGE LINES AND GIVE ADDED STABILITY

SECTION 2-5

2-FT-HIGH EARTH FILLED SACKS*



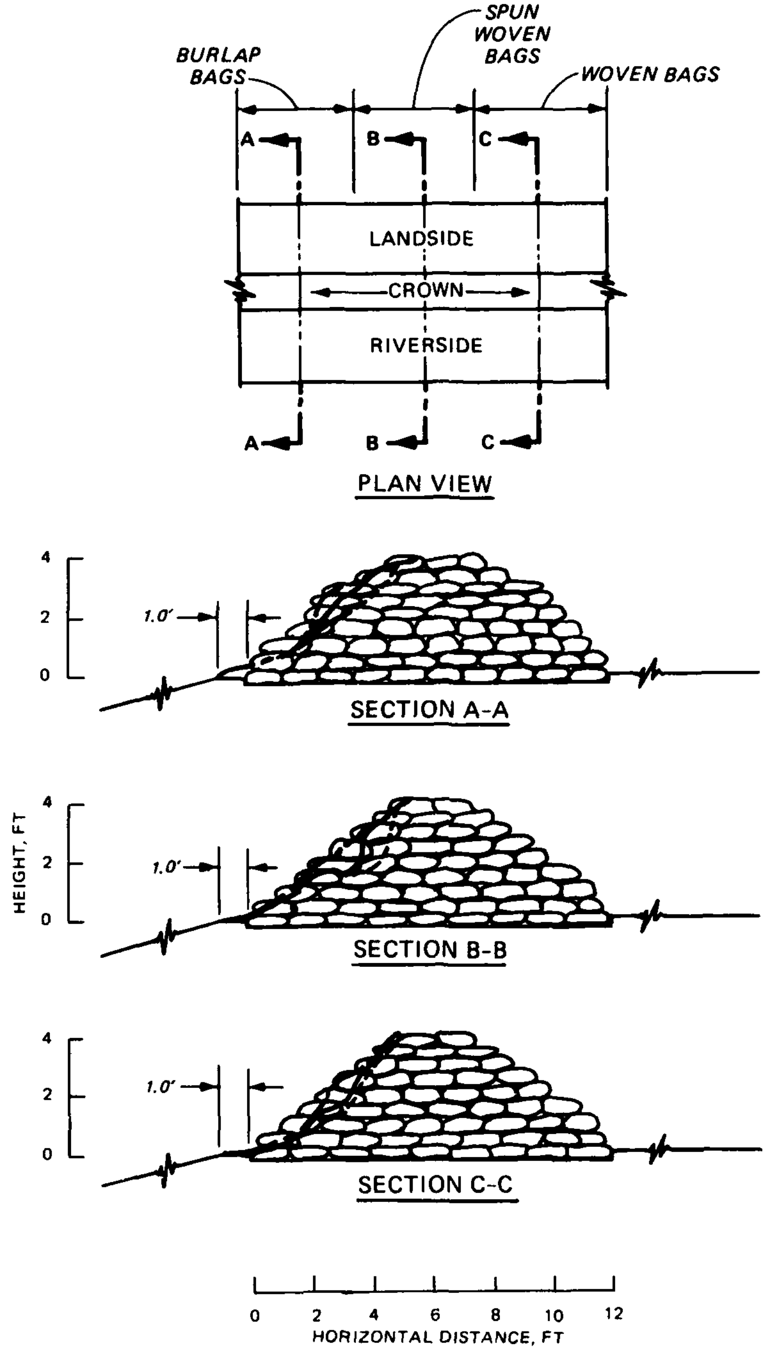
SECTION 2-5
BEFORE AND AFTER TEST CROSS SECTIONS (ESTIMATED)



*SACKS ARE ONE-HALF TO TWO-THIRDS FULL AND ARE UNTIED. UNTIED SACK ENDS ARE FOLDED BACK AND ADJACENT SACKS CLOSED ENDS ARE LAYED OVER FOLDS

**NOTE OFFSET OF SEAMS TO BREAK SEEPAGE LINES AND GIVE STABILITY

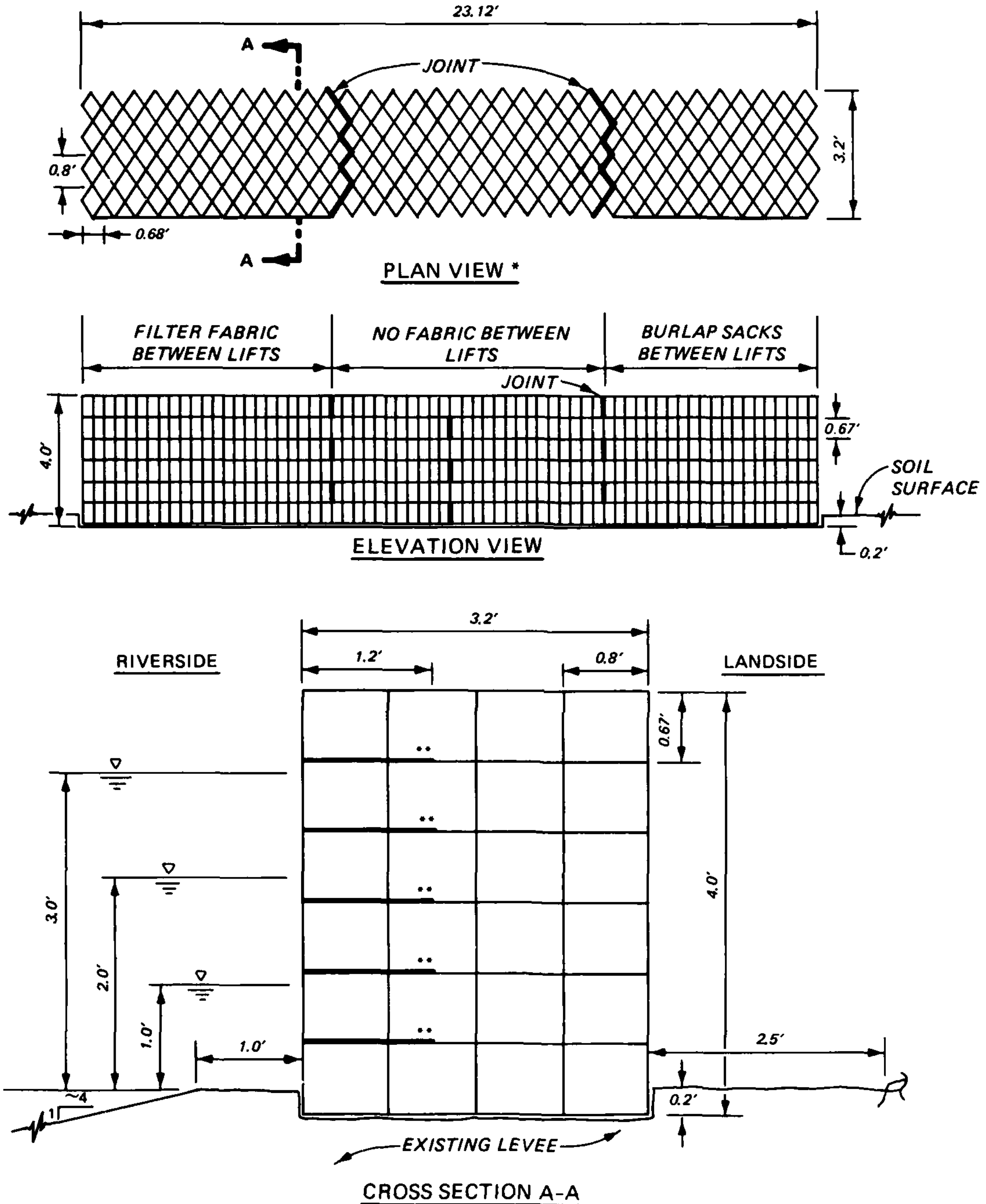
SECTION 4-1
4-FT-HIGH EARTH-FILLED SACKS*



LEGEND

- AFTER 19 HR OF T=1.0 SEC H ≅ 0.75 FT @ 1.0 FT WATER DEPTH (TOTAL TIME OF 264 HR AT WATER DEPTH)
- AFTER 19 HR OF T=1.0 SEC H ≅ 0.75 FT @ 2.0 FT WATER DEPTH (TOTAL TIME OF 640 HR AT WATER DEPTH)
- AFTER 8 HR OF T = 1.0 SEC H ≅ 0.75 FT @ 3.0 FT WATER DEPTH (TOTAL TIME OF 116 HR AT WATER DEPTH)

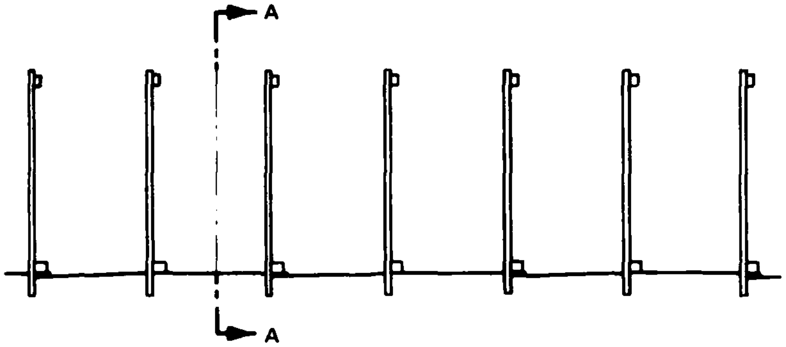
SECTION 4-1
BEFORE AND AFTER TEST CROSS SECTIONS (ESTIMATED)



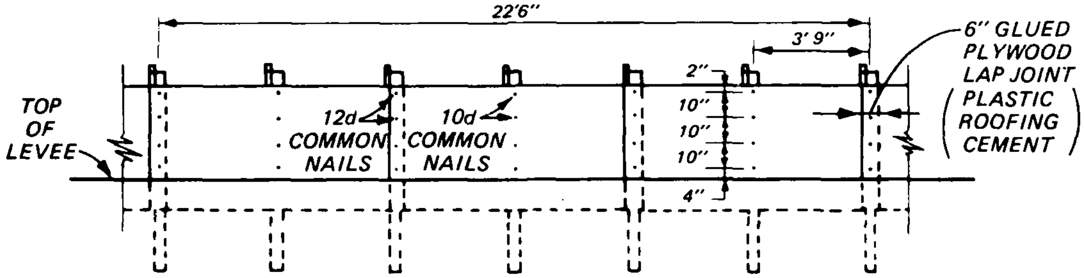
* CELL IDEALIZED AS DIAMOND; ACTUAL EXPANDED SHAPE IS

**EXTENT OF BURLAP OR FILTER FABRIC WHERE USED

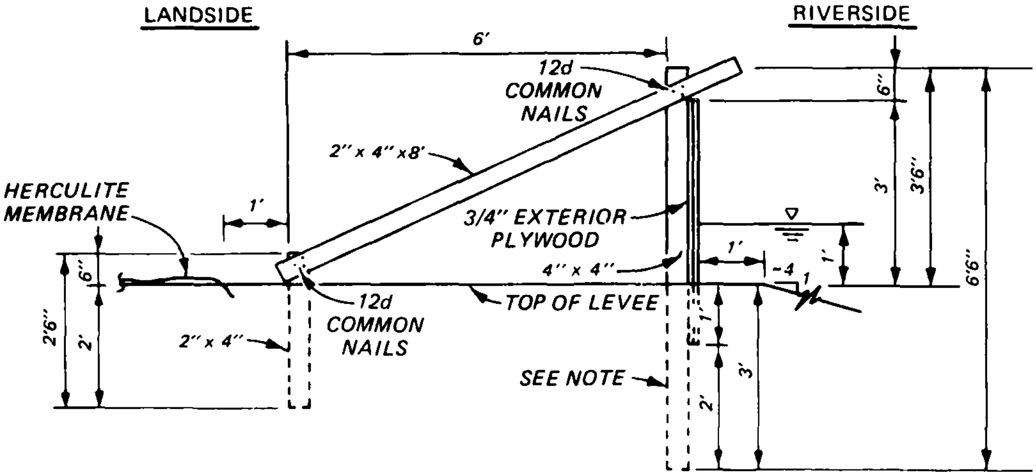
SECTION 4-2
4-FT-HIGH PLASTIC GRID
WITH
SAND FILL



PLAN



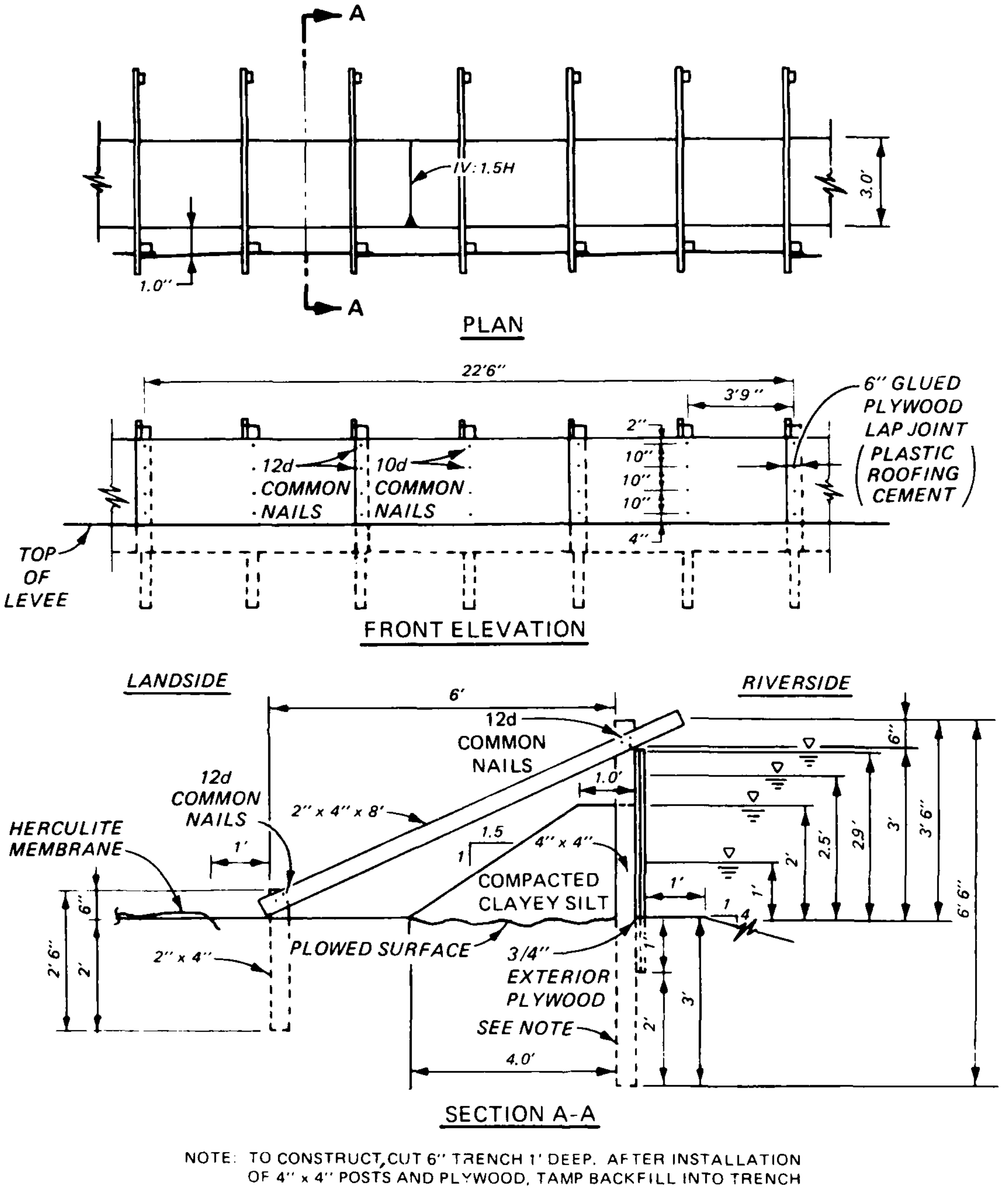
FRONT ELEVATION



SECTION A-A

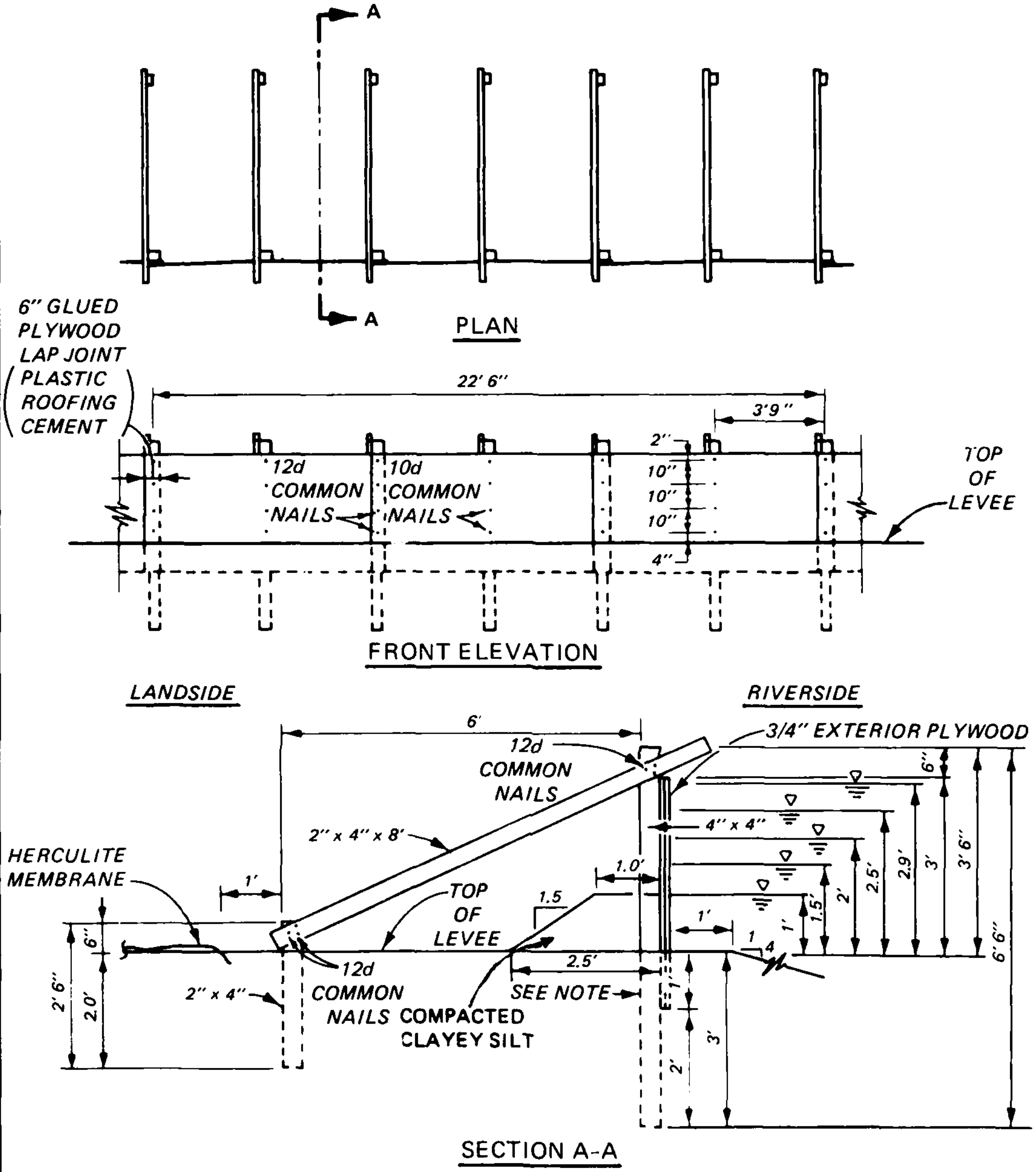
*NOTE: TO CONSTRUCT, CUT 6" TRENCH 1' DEEP. AFTER INSTALLATION OF 4" x 4" POSTS AND PLYWOOD, TAMP BACKFILL INTO TRENCH

SECTION 4-3
PLYWOOD FLASHBOARD



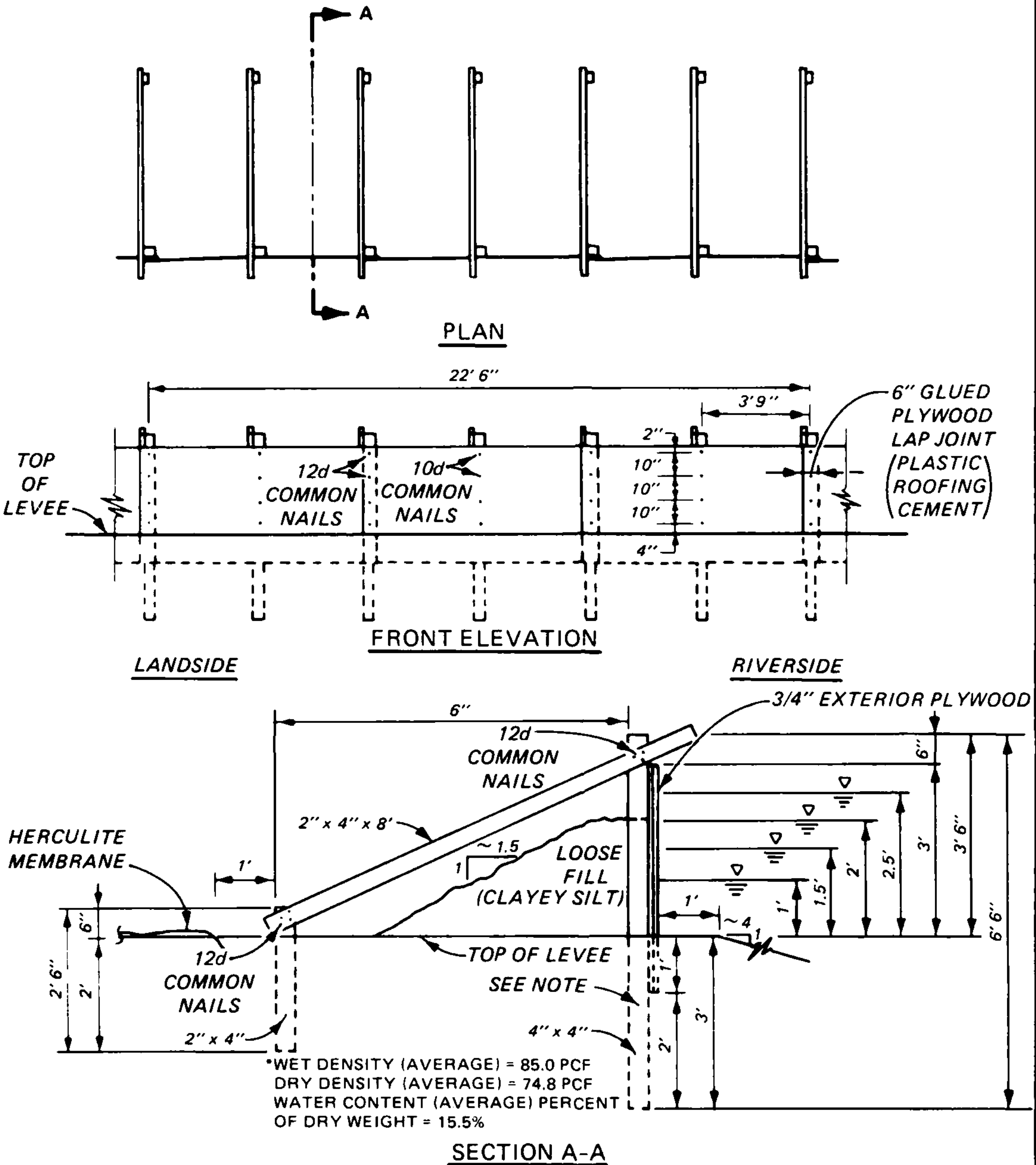
SECTION 4-3-A

PLYWOOD FLASHBOARD WITH 2-FT-HIGH COMPACTED EARTH BACKING



NOTE: TO CONSTRUCT CUT 6" TRENCH 1' DEEP. AFTER INSTALLATION OF 4" x 4" POSTS AND PLYWOOD, TAMP BACKFILL INTO TRENCH

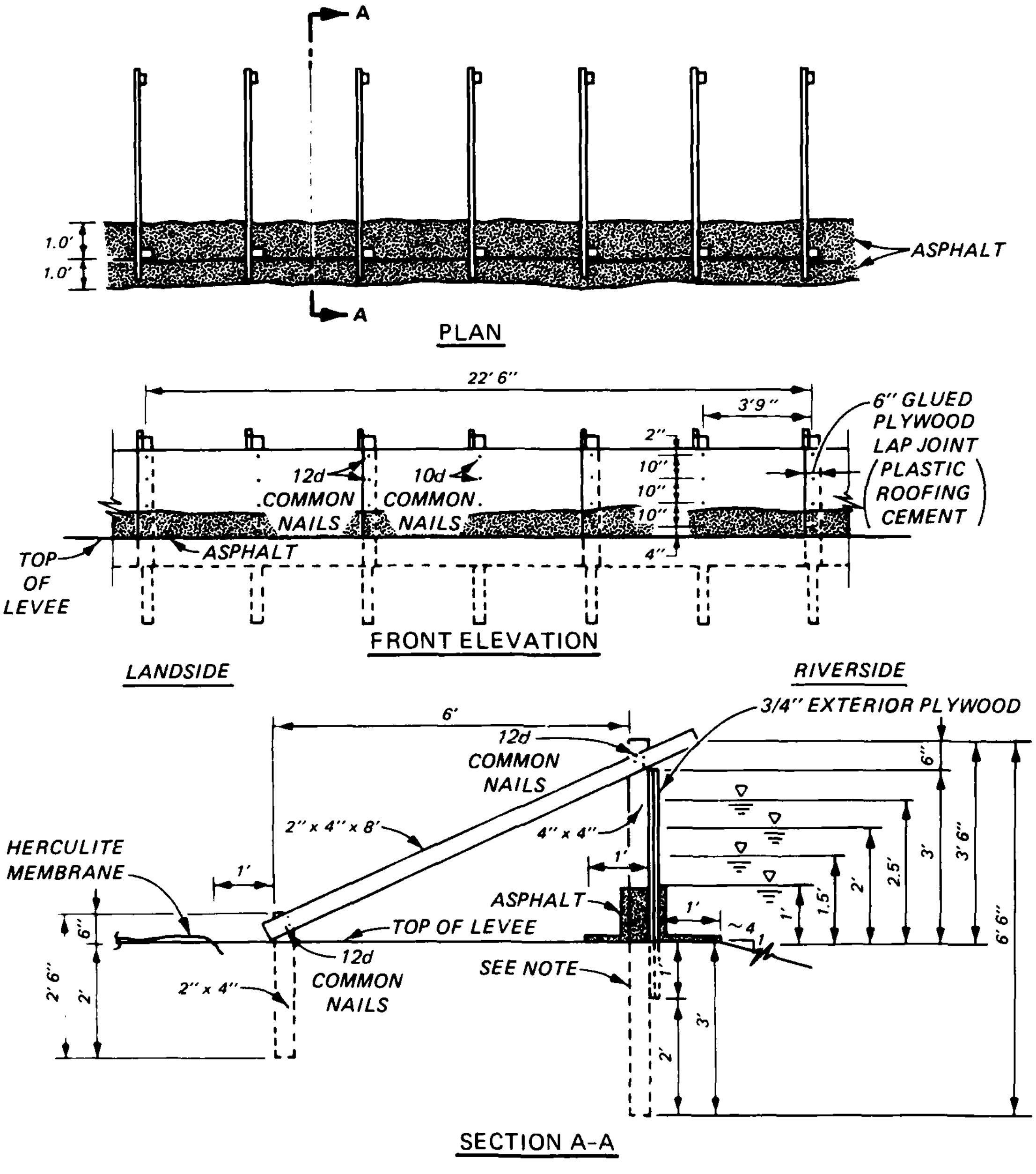
SECTION 4-3-B
PLYWOOD FLASHBOARD WITH 1-FT-HIGH COMPACTED EARTH BACKING



NOTE: TO CONSTRUCT, CUT 6" TRENCH 1' DEEP. AFTER INSTALLATION OF 4" x 4" POSTS AND PLYWOOD, TAMP BACKFILL INTO TRENCH

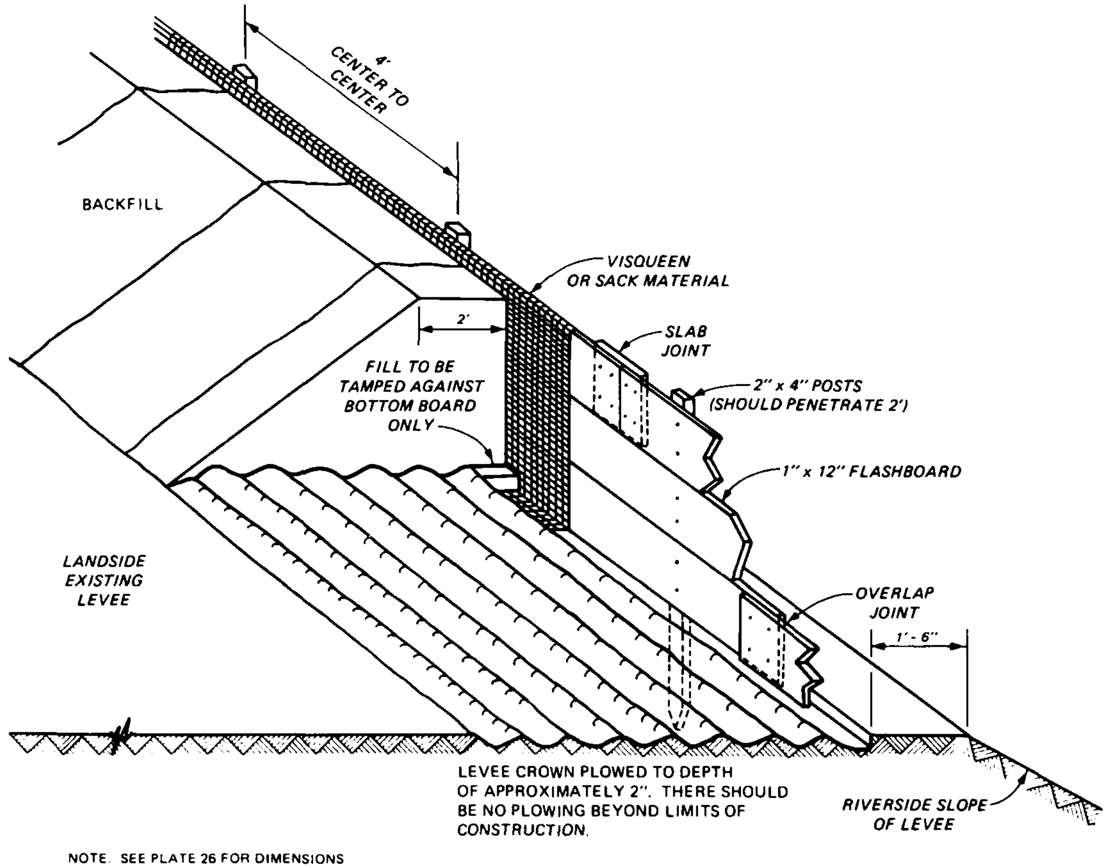
SECTION 4-3-C

PLYWOOD FLASHBOARD WITH 2-FT-HIGH UNCOMPACTED EARTH BACKING

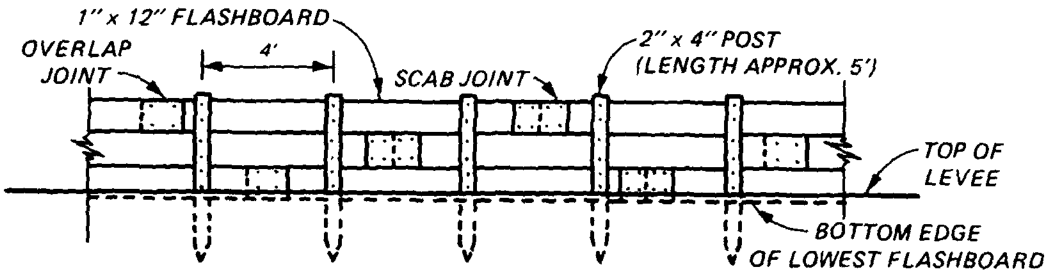


NOTE: TO CONSTRUCT, CUT 6" TRENCH 1' DEEP. AFTER
OF 4" x 4" POSTS AND PLYWOOD, TAMP BACKFILL INTO TRENCH

SECTION 4-3-D
**PLYWOOD FLASHBOARD
WITH
ASPHALT SEALER**

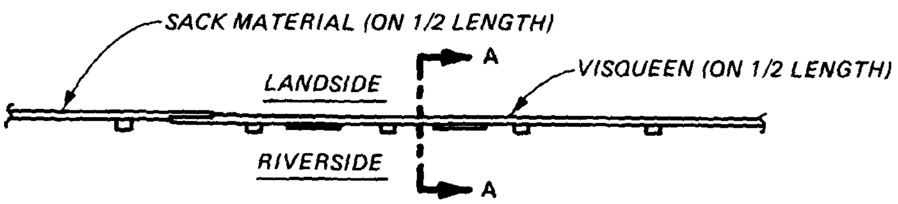


SECTION 4-4
PLANKING FLASHBOARD
WITH
EARTH BACKING

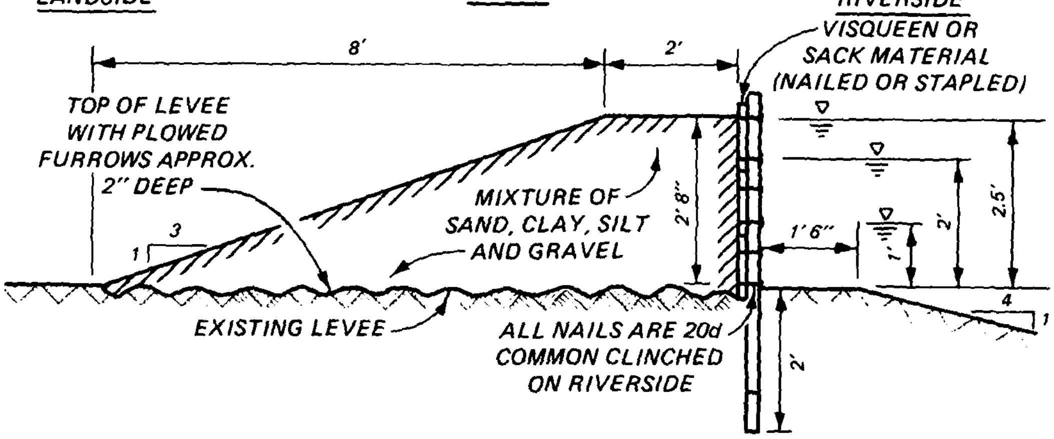


RIVERSIDE ELEVATION

NOTES:
1. TEST SECTION LENGTH APPROXIMATELY 24 FT.
2. FOR FLASHBOARD BACKING, 1/2 THE LENGTH EQUIPPED WITH SACK MATERIAL AND 1/2 WITH VISQUEEN.
3. ALL DIMENSIONS ARE FOR ROUGH CUT LUMBER

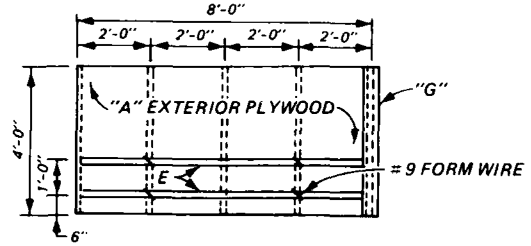


PLAN

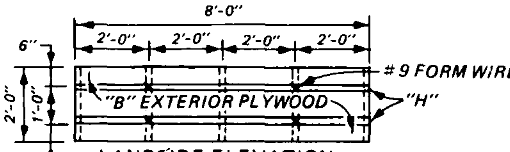


SECTION A-A

SECTION 4-4
3-FT-HIGH
PLANKING FLASHBOARD WITH
EARTH BACKING



RIVERSIDE ELEVATION

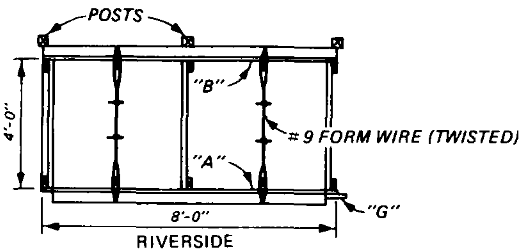


LANDSIDE ELEVATION

MATERIALS LIST PER 8 FT. SECTION

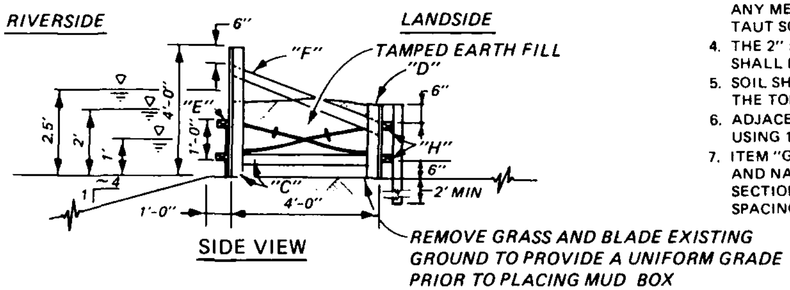
MARK NO	NO OF PIECES	DIMENSION	UNIT LENGTH	TOTAL LENGTH	NO 4' x 8' SHEETS
"A"	1	5/8" OR 3/4" x 4'-0"	8'	8'	1
"B"	1	5/8" OR 3/4" x 2'-0"	8'	8'	1/2
"C"	8	2" x 4"	4'	32'	- -
"D"	5	2" x 4"	2'	10'	- -
"E"	2	2" x 4"	7'-6"	15'	- -
"F"	3	2" x 4"	4'-6"	13'-6"	- -
POST	2	4" x 4"	4'-0"	8'-0"	- -
"G"	1	5/8" OR 3/4" x 6"	4'-0"	4'-0"	1/16
"H"	2	2" x 4"	7'-11"	16'	- -

10D NAILS (COMMON) = 1.6 LBS
16D NAILS (COMMON) = 1.0 LBS
#9 FORM WIRE 48'



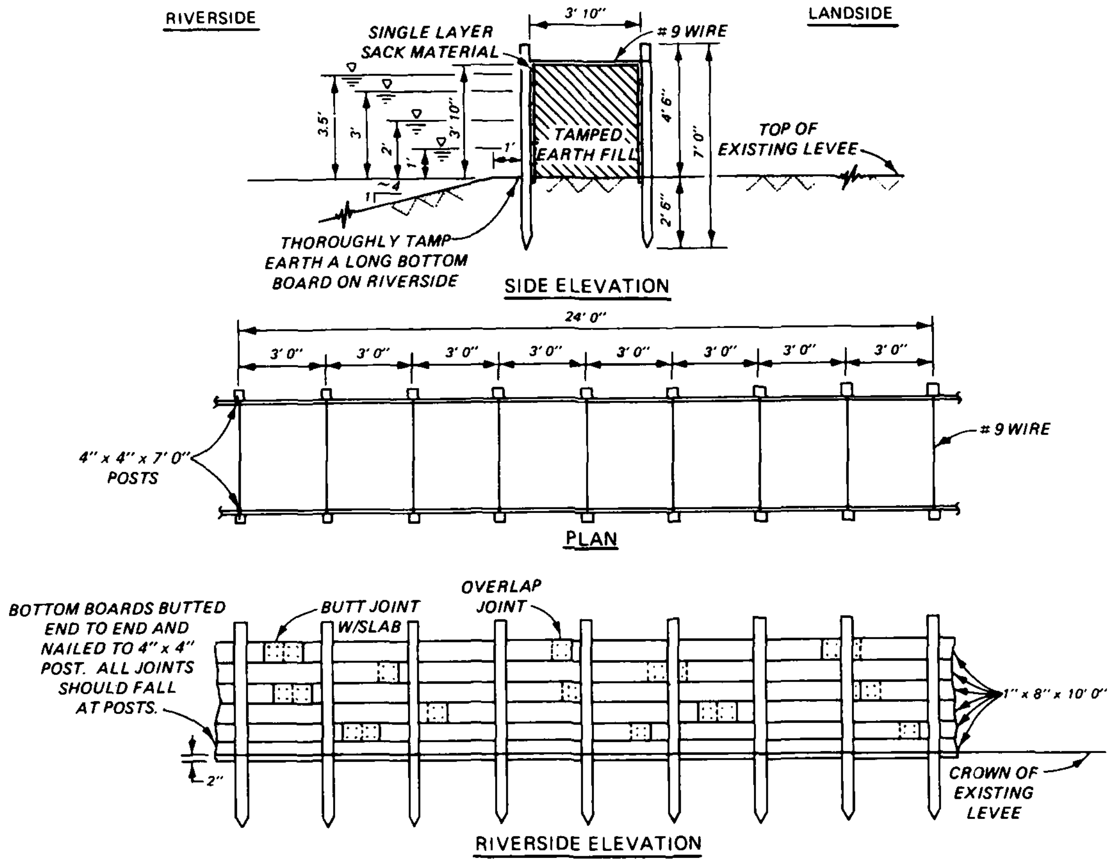
RIVERSIDE
PLAN VIEW

NOTES:
1. ALL PLYWOOD SHALL BE NAILED TO 2" x 4" MEMBERS WITH 10 PENNY NAILS AT A MAXIMUM SPACING OF 12 INCHES ON CENTER.
2. ALL 2" x 4" TIMBERS SHALL BE NAILED TO LIKE TIMBERS WITH 3-16 PENNY NAILS AT EACH CONNECTION.
3. #9 FORM WIRES ARE TO BE FASTENED TO 2" x 4" MEMBER BY ANY METHOD APPROVED BY CONTRACTING OFFICER AND PULLED TAUT SO THERE IS NO SAG.
4. THE 2" x 4" MEMBER THAT THE WIRE IS FASTENED TO SHALL BE NAILED TO PANELS "A" AND "B".
5. SOIL SHALL BE BACKFILLED TO AN ELEVATION LEVEL WITH THE TOP OF PANEL "A"
6. ADJACENT MUD BOX SECTIONS ARE TO BE NAILED TOGETHER USING 16 PENNY NAILS SPACED AT 4-INCH MAXIMUM SPACING.
7. ITEM "G" IS TO BE COATED WITH PLASTIC ROOFING CEMENT AND NAILED ACROSS BUTT JOINT BETWEEN ADJACENT 8' SECTIONS USING 10 PENNY NAILS ON A MAXIMUM OF 6-INCH SPACING.



SIDE VIEW

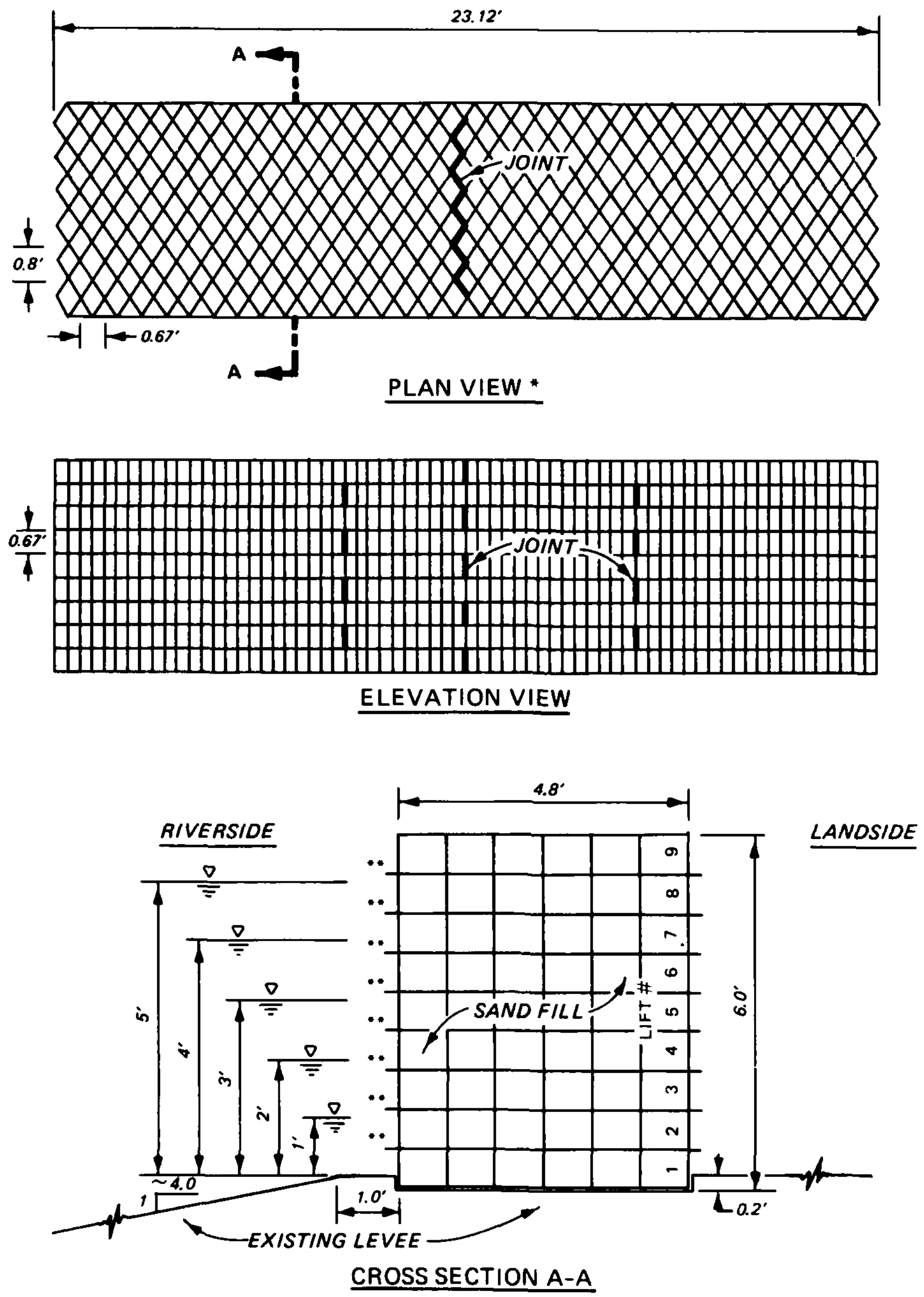
SECTION 4-5
4-FT-HIGH PLYWOOD MUD BOX WITH EARTH FILL



NOTES. 1. ALL BOARDS NAILED TO POSTS AND BUTT AND OVERLAP JOINTS NAILED USING 8d COMMON NAILS
2. SOD SHOULD BE REMOVED FROM AREA WHERE MUD BOX WILL BE CONSTRUCTED AND BOTTOM RIVERSIDE AND LANDSIDE BOARDS SHOULD EXTEND AT LEAST 2" INTO THE EXISTING LEVEE.

* ON SECTION 4-6-A, EVERY OTHER PAIR OF SUPPORT POSTS WERE REMOVED MAKING A 6-FT SPACING BETWEEN SUPPORT POSTS

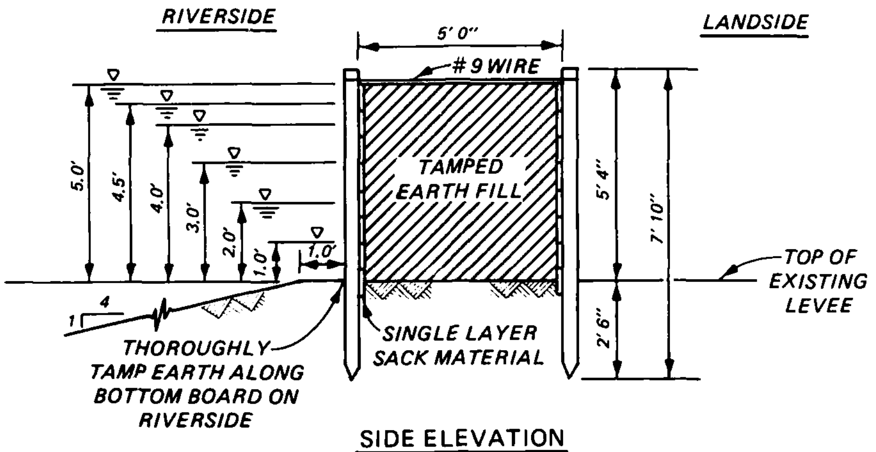
SECTION 4-6 AND 4-6-A*
4-FT-HIGH PLANKING MUD BOX WITH TAMPED EARTH FILL



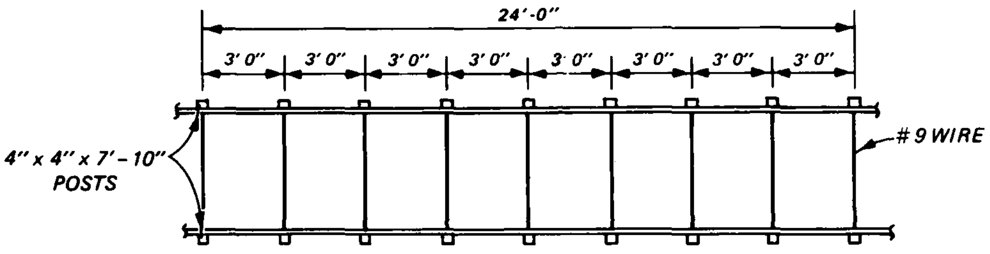
* CELL IDEALIZED AS DIAMOND; ACTUAL EXPANDED SHAPE IS 0.8' 0.67'

**FILTER FABRIC PLACED OVER ENTIRE WIDTH BETWEEN LIFTS

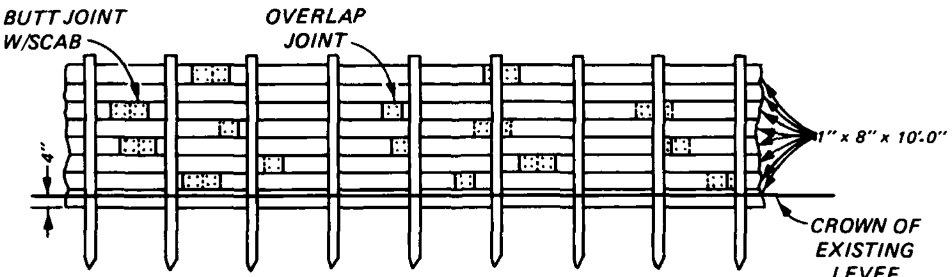
SECTION 6-1
6-FT-HIGH PLASTIC GRID WITH SAND FILL



SIDE ELEVATION



PLAN

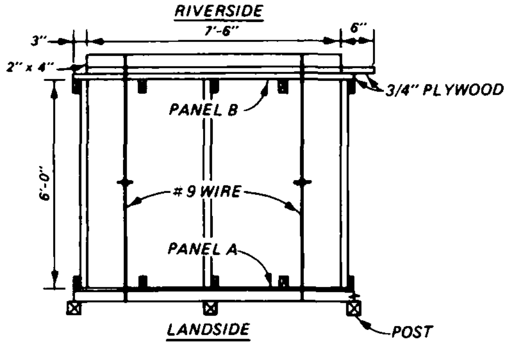


RIVERSIDE ELEVATION

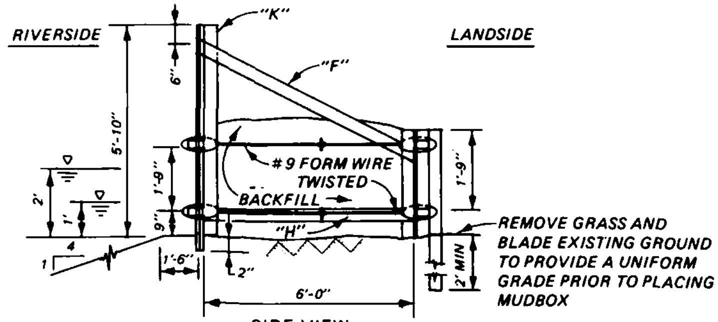
NOTES:
1. ALL BOARDS NAILED TO POSTS AND BUTT AND LAP JOINTS NAILED USING 8d COMMON NAILS.
2. SOD SHOULD BE REMOVED FROM AREA WHERE MUD BOX WILL BE CONSTRUCTED AND BOTTOM RIVERSIDE AND LANDSIDE BOARDS SHOULD EXTEND AT LEAST 4 INCHES INTO EXISTING LEVEE
3. BOTTOM BOARDS SHOULD BUTT END TO END AND JOINTS SHOULD FALL AT AND BE NAILED TO POSTS.

SECTION 6-2

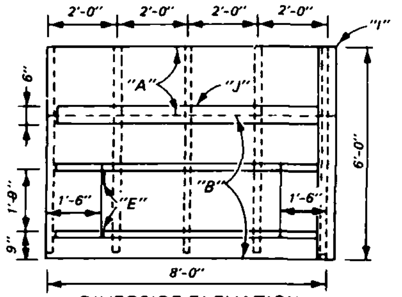
5-FT-HIGH PLANKING MUD BOX WITH TAMPED EARTH FILL



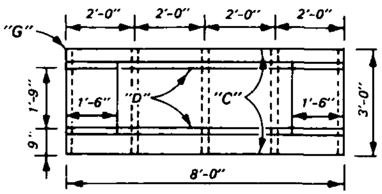
PLAN VIEW



SIDE VIEW



RIVERSIDE ELEVATION



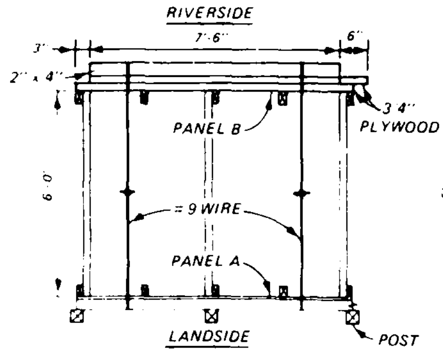
LANDSIDE ELEVATION

MATERIALS LIST PER 8 FT. SECTION

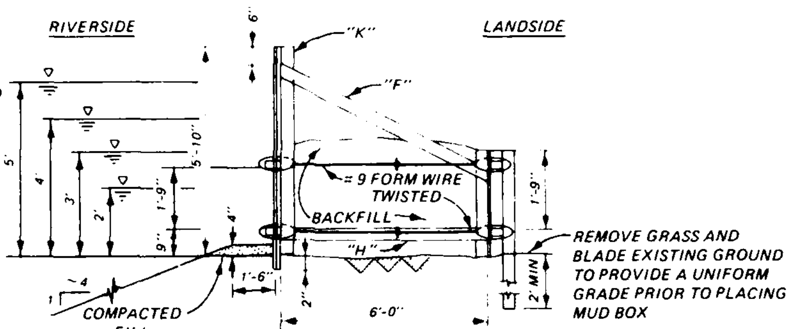
TIMER MARK	NUMBER OF PIECES	DIMENSION	UNIT LENGTH	TOTAL LENGTH
"A"	1	2'-0" x 3/4"	8'-0"	8'-0"
"B"	1	4'-0" x 3/4"	8'-0"	8'-0"
"C"	1	3'-0" x 3/4"	8'-0"	8'-0"
"D"	2	2' x 4"	8'-0"	16'-0"
"E"	2	2" x 4"	7'-6"	15'-0"
"F"	3	2" x 4"	6'-8 1/2"	20'-1 1/2"
"G"	5	2" x 4"	3'-0"	15'-0"
POST	2	4" x 4"	5'-0"	10'-0"
"H"	3	2" x 4"	6'-0"	18'-0"
"I"	1	6" x 3/4"	6'-0"	6'-0"
"J"	1	6" x 3/4"	7'-6"	7'-6"
"K"	5	2" x 4"	5'-10"	29'-2"

NOTES:
1. ALL PLYWOOD SHALL BE NAILED TO 2" x 4" MEMBERS WITH 10 PENNY NAILS AT A MAXIMUM SPACING OF 12 INCHES ON CENTER
2. ALL 2" x 4" TIMBERS SHALL BE NAILED TO LIKE TIMBERS WITH 4-16 PENNY NAILS AT EACH CONNECTION
3. #9 FORM WIRES ARE TO BE FASTENED TO 2" x 4" MEMBER BY ANY METHOD APPROVED BY CONTRACTING OFFICER AND PULLED TAUT SO THERE IS NO SAG
4. THE 2" x 4" MEMBER THAT THE WIRE IS FASTENED TO SHALL BE NAILED TO PANELS "A" AND "B"
5. SOIL SHALL BE BACKFILLED TO AN ELEVATION LEVEL WITH THE TOP OF PANEL "A"
6. USE OF 3 PIECES OF 12" x 3/4" x 8'-0" PLYWOOD MAY BE USED IN PLACE OF 1 PIECE OF 3'-0" x 3/4" x 8'-0" PLYWOOD FOR CONSTRUCTION OF PANEL "A"
7. ADJACENT MUD BOX SECTIONS ARE TO BE NAILED TOGETHER USING 16 PENNY NAILS SPACED AT 4-INCH MAXIMUM SPACING
8. PANEL "B" SHALL BE DRIVEN INTO 2 INCH DEEP SLIT TRENCH CUT IN EXISTING LEVEE
9. ITEMS "I" AND "J" SHALL BE ATTACHED TO PANEL "B" WITH ROOFING CEMENT AND NAILED AT A MINIMUM OF 6 INCH SPACING

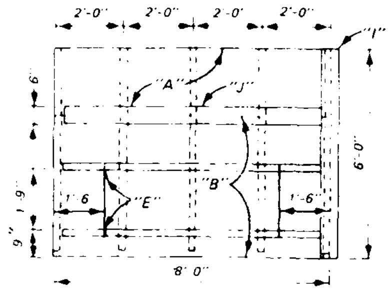
SECTION 6-3
6-FT-HIGH PLYWOOD MUD BOX WITH EARTH FILL



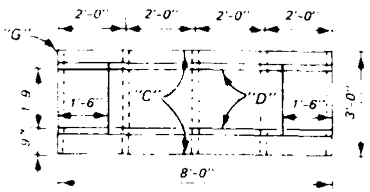
PLAN VIEW



SIDE VIEW



RIVERSIDE ELEVATION



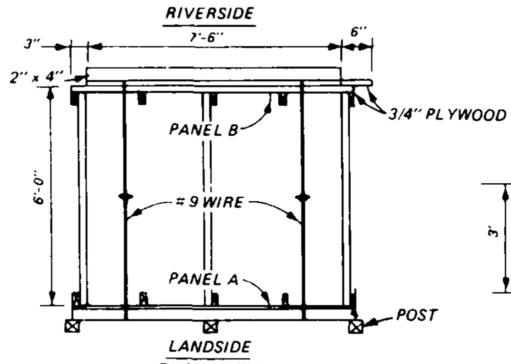
LANDSIDE ELEVATION

MATERIALS LIST PER 8 FT SECTION

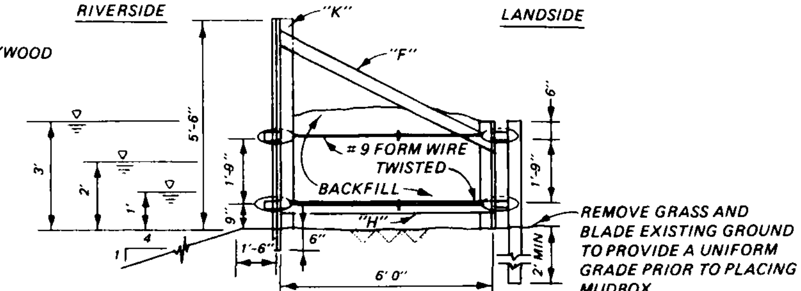
TIMBER MARK	NUMBER OF PIECES	DIMENSION	UNIT LENGTH	TOTAL LENGTH
"A"	1	2'-0" x 3/4"	8'-0"	8'-0"
"B"	1	4'-0" x 3/4"	8'-0"	8'-0"
"C"	1	3'-0" x 3/4"	8'-0"	8'-0"
"D"	2	2" x 4"	8'-0"	10'-0"
"E"	2	2" x 4"	7'-6"	15'-0"
"F"	3	2" x 4"	6'-8 1/2"	20'-1 1/2"
"G"	5	2" x 4"	3'-0"	15'-0"
POST	2	4" x 4"	5'-0"	10'-0"
"H"	3	2" x 4"	6'-0"	18'-0"
"I"	1	6" x 3/4"	6'-0"	6'-0"
"J"	1	6" x 3/4"	7'-6"	7'-6"
"K"	5	2" x 4"	5'-10"	29'-2"

NOTES
1. ALL PLYWOOD SHALL BE NAILED TO 2" x 4" MEMBERS WITH 10 PENNY NAILS AT A MAXIMUM SPACING OF 12 INCHES ON CENTER
2. ALL 2" x 4" TIMBERS SHALL BE NAILED TO LIKE TIMBERS WITH 4-16 PENNY NAILS AT EACH CONNECTION
3. =9 FORM WIRES ARE TO BE FASTENED TO 2" x 4" MEMBER BY ANY METHOD APPROVED BY CONTRACTING OFFICER AND PULLED TAUT SO THERE IS NO SAG
4. THE 2" x 4" MEMBER THAT THE WIRE IS FASTENED TO SHALL BE NAILED TO PANELS "A" AND "B"
5. SOIL SHALL BE BACKFILLED TO AN ELEVATION LEVEL WITH THE TOP OF PANEL "A"
6. USE OF 3 PIECES OF 12" x 3/4" x 8'-0" PLYWOOD MAY BE USED IN PLACE OF 1 PIECE OF 3'-0" x 3/4" x 8'-0" PLYWOOD FOR CONSTRUCTION OF PANEL "A"
7. ADJACENT MUD BOX SECTIONS ARE TO BE NAILED TOGETHER USING 16 PENNY NAILS SPACED AT 4 INCH MAXIMUM SPACING
8. PANEL B SHALL BE DRIVEN INTO 2 INCH DEEP SLIT TRENCH CUT IN EXISTING LEVEE
9. ITEMS "I" AND "J" SHALL BE ATTACHED TO PANEL F WITH ROOFING CEMENT AND NAILED AT A MINIMUM OF SIX INCH SPACING
10. EXCEPT FOR THE 4 INCH OF COMPACTED FILL PLACED ON THE RIVERSIDE OF THIS SECTION, IT IS IDENTICAL TO SECTION 6-3

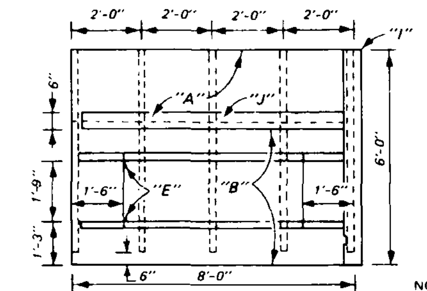
SECTION 6-3-A
6-FT-HIGH PLYWOOD MUD BOX WITH EARTH FILL AND RIVERSIDE BERM



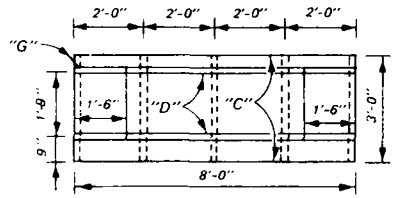
PLAN VIEW



SIDE VIEW



RIVERSIDE ELEVATION



LANDSIDE ELEVATION

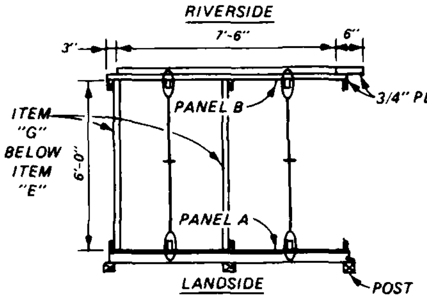
MATERIALS LIST PER 8 FT. SECTION

TIMBER MARK	NUMBER OF PIECES	DIMENSION	UNIT LENGTH	TOTAL LENGTH
"A"	1	2'-0" x 3/4"	8'-0"	8'-0"
"B"	1	4'-0" x 3/4"	8'-0"	8'-0"
"C"	1	3'-0" x 3/4"	8'-0"	8'-0"
"D"	2	2" x 4"	8'-0"	10'-0"
"E"	2	2" x 4"	7'-6"	15'-0"
"F"	3	2" x 4"	6'-8 1/2"	20'-1 1/2"
"G"	5	2" x 4"	3'-0"	15'-0"
POST	2	4" x 4"	5'-0"	10'-0"
"H"	3	2" x 4"	6'-0"	18'-0"
"I"	1	6" x 3/4"	6'-0"	6'-0"
"J"	1	6" x 3/4"	7'-6"	7'-6"
"K"	5	2" x 4"	5'-6"	27'-6"

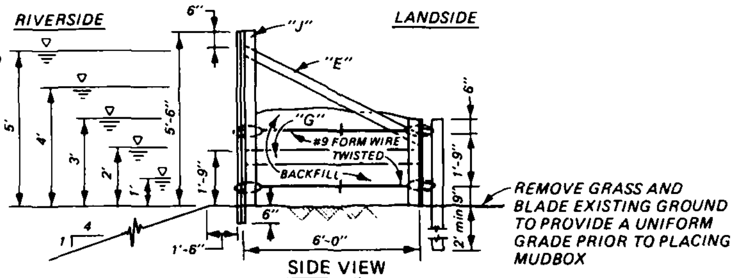
NOTES.

1. ALL PLYWOOD SHALL BE NAILED TO 2" x 4" MEMBERS WITH 10 PENNY NAILS AT A MAXIMUM SPACING OF 12 INCHES ON CENTER
2. ALL 2" x 4" TIMBERS SHALL BE NAILED TO LIKE TIMBERS WITH 4-16 PENNY NAILS AT EACH CONNECTION
3. #9 FORM WIRES ARE TO BE FASTENED TO 2" x 4" MEMBER BY ANY METHOD APPROVED BY CONTRACTING OFFICER AND PULLED TAUT SO THERE IS NO SAG
4. THE 2" x 4" MEMBER THAT THE WIRE IS FASTENED TO SHALL BE NAILED TO PANELS "A" AND "B"
5. SOIL SHALL BE BACKFILLED TO AN ELEVATION LEVEL WITH THE TOP OF PANEL "A"
6. USE OF 3 PIECES OF 12" x 3/4" x 8'-0" PLYWOOD MAY BE USED IN PLACE OF 1 PIECE OF 3'-0" x 3/4" x 8'-0" PLYWOOD FOR CONSTRUCTION OF PANEL "A"
7. ADJACENT MUD BOX SECTIONS ARE TO BE NAILED TOGETHER USING 16 PENNY NAILS SPACED AT 4-INCH MAXIMUM SPACING
8. ITEMS "I" AND "J" SHALL BE ATTACHED TO PANEL B WITH ROOFING CEMENT AND NAILED AT A MAXIMUM OF SIX INCH SPACING.
9. CUT TRENCH SIX INCH DEEP AND PLACE BOTTOM SIX INCH OF PANEL B AGAINST RIVERSIDE EDGE OF TRENCH AND COMPACT FILL BACK INTO TRENCH. KEEP TRENCH AS NARROW AS POSSIBLE.

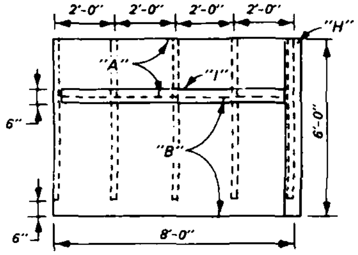
SECTION 6-4
6-FT-HIGH ENTRENCHED PLYWOOD MUD BOX WITH EARTH FILL



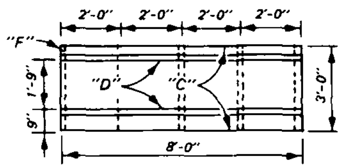
PLAN VIEW



SIDE VIEW



RIVERSIDE ELEVATION



LANDSIDE ELEVATION

MATERIALS LIST PER 8 FT SECTION

TIMBER MARK	NUMBER OF PIECES	DIMENSION	UNIT LENGTH	TOTAL LENGTH
"A"	1	2'-0" x 3/4"	8'-0"	8'-0"
"B"	1	4'-0" x 3/4"	8'-0"	8'-0"
"C"	1	3'-0" x 3/4"	8'-0"	8'-0"
"D"	2	2" x 4"	8'-0"	16'-0"
"E"	3	2" x 4"	6'-8-1/2"	20'-1-1/2"
"F"	5	2" x 4"	3'-0"	15'-0"
POST	2	4" x 4"	5'-0"	10'-0"
"G"	2	2" x 4"	6'-0"	18'-0"
"H"	1	6" x 3/4"	6'-0"	6'-0"
"I"	1	6" x 3/4"	7'-6"	7'-6"
"J"	5	2" x 4"	5'-6"	27'-6"

NOTES:
1. ALL PLYWOOD SHALL BE NAILED TO 2" x 4" MEMBERS WITH 10 PENNY NAILS AT A MAXIMUM SPACING OF 12 INCHES ON CENTER
2. ALL 2" x 4" TIMBERS SHALL BE NAILED TO LIKE TIMBERS WITH 4-16 PENNY NAILS AT EACH CONNECTION.
3. #9 FORM WIRES ARE TO BE FASTENED TO 2" x 4" MEMBER BY ANY METHOD APPROVED BY CONTRACTING OFFICER AND PULLED TAUT SO THERE IS NO SAG.
4. THE 2" x 4" MEMBER THAT THE WIRE IS FASTENED TO SHALL BE NAILED TO PANEL "A"
5. SOIL SHALL BE BACKFILLED TO AN ELEVATION LEVEL WITH THE TOP OF PANEL "A"
6. USE OF 3 PIECES OF 12" x 3/4" x 8'-0" PLYWOOD MAY BE USED IN PLACE OF 1 PIECE OF 3'-0" x 3/4" x 8'-0" PLYWOOD FOR CONSTRUCTION OF PANEL "A"
7. ADJACENT MUD BOX SECTIONS ARE TO BE NAILED TOGETHER USING 16 PENNY NAILS SPACED AT 4-INCH MAXIMUM SPACING
8. *CUT TRENCH 6 IN. DEEP AND PLACE BOTTOM 6 IN. OF* PANEL "B" IN TRENCH AND THOROUGHLY COMPACT BACK FILL
9. ITEMS "H" AND "I" SHALL BE ATTACHED TO PANEL "B" WITH ROOFING CEMENT AND NAILED AT A MAXIMUM OF 6 IN. SPACING

SECTION 6-5
6-FT-HIGH
ENTRENCHED MUD BOX
WITH
EARTH FILL

END
DATE
FILMED
8-88
DTIC